# Mass movement and coastal cliff development of the Isle of Purbeck, Dorset.

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# MASS MOVEMENT AND COASTAL CLIFF DEVELOPMENT

# OF THE ISLE OF PURBECK, DORSET.

Vol II

by

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# Chapter V FIELD INVESTIGATIONS

## 5.1 INTRODUCTION

Of major importance to this study is the field instrumentation and monitoring of mass movements, to examine in detail interrelationships between variables governing stability. Due to the nature of failures in different types of material this, is only possible for the Wealden Beds. Rockfalls in hard brittle materials, such as the Portland Limestone and Chalk, are high magnitude - low frequency events which do not lend themselves to such examination. Investigations were conducted to provide empirical data on rates of movement and details of the resultant morphological effects, identify the spatial characteristics of movements and assess the reasons for this, consider the relationship between mapped geomorphology and the differing rates of movement, identify relationships between material geotechnical properties and movement rates, define the effects and importance of parameters thought to govern slope movements (Anderson & Richards, 1981; Bhandari & Hutchinson, 1982; Early and Skempton, 1972; Hutchinson & Bhandari, 1971) and to provide details required for slope stability analysis.

The time available for data collection here was restricted both in terms of the length of the field period and also the frequency of visits to the field sites. The need for long term records in mass movement studies is frequently recognised (Brunsden & Thornes, 1977; Chorley et al., 1984; Cullingford et al., 1980; Thornes, 1982). Records extend from September 1983 to May 1985 for two reasons. Readings were kept over two winter periods when instability is at its greatest. Some indication of medium term (> 1 year) variations in mass movement rates were therefore established. Monthly site visits from September 1983 to September 1984 were increased to a weekly frequency thereafter. These frameworks provided representative data, relative to the type of equipment in use during each monitoring phase.

The purchase of new equipment specifically for this study was limited. This was overcome in three ways. Apparatus available at King's College London, which had been used for previous research projects was modified. Equipment was borrowed from other Universities and Institutes (Table 5.1) and new apparatus was designed and constructed from individual components, keeping costs to a minimum. The type of apparatus used was restricted by site conditions. Geomorphological mapping suggests that many of the slides are little more than 1.0 - 1.5 m deep in places, this dictating the specifications of much of the equipment. Small changes in porewater pressure across a slip surface which is close to the ground surface, for example, can significantly alter the stability of the slope. In this instance, monitoring of the phreatic surface must therefore be sensitive to small changes in water level and quickly responsive to alterations in field conditions. Also, visitor pressure and the impracticalities of enclosing apparatus required all equipment to be either buried or flush with the ground surface.

The chosen techniques include those used to quantify site characteristics important to field instrumentation, standard apparatus used for the main data collection and electronic equipment installed at one site for detailed investigations.

TABLE 5.1

Equipment obtained for use with this study

| Equipment | Use to this Study | Supplier |
| --- | --- | --- |
| Hunter Seismograph | Location of slip surfaces in mudslides | Geology Dept. University College London |
| Electronic distance measuring & survey | Measuring movement via pin network | Photogrammetry & Surveying, U.C.L. |
| Peekel Micro-strain gauge & transducer | Measurement of porewater pressure | Civil Engineering Imperial College |
| Epsylon logger & shape pressure transducer | Measurement of porewater pressure | Institute of Hydrology |
| Continuous recording tipping bucket rain gauge | Monitoring of precipitation | Birkbeck College London |
| Digital event recorder | Continuous monitoring of precipitation | Institute of Hydrology |
| Automatic weather station | Recording climate | N.E.R.C. equipment pool |
| Neutron probe | Soil moisture recording | Institute of Hydrology |
| Data loggers | Continuously recording movement | London School of Economics |
| Data logger components | Continously recording movement | Microdata Ltd, Radlett, Herts |

## 5.2 LOCATION OF INSTRUMENTATION

The geomorphological map was used to identify mass movements suitable for instrumentation. Mudslides were chosen at points along the coast. Each was required to display a discrete morphology with identifiable rear supply area, track and toe lobe, while also being representative of overall site conditions defined by the geotechnical study. The extremities of each feature had to display a distinct lateral shear, extending from slope crest to toe along a slide of reasonable cross-sectional uniformity. Locations were also chosen to minimise human interference and the effects of footpaths, drainage culverts and the trampling of equipment. These criteria were used to select four mudslides for instrumentation, one each at Durdle Door, Stair Hole, Mupe Bay and Worbarrow Bay.

## 5.3 PRELIMINARY SITE INVESTIGATIONS

Although "field investigation is the central and decisive part of a study of landslides and landslide prone areas" (Philbrick & Cleves, 1958), the success of the study relies heavily on initial ground investigations and the careful selection of apparatus. The merits of different field techniques have been widely considered (American Society of Civil Engineers, 1972; British Geotechnical Society, 1974; Brunsden, 1984; Shannon et al., 1962; Wilson & Mikkelsen, 1978). Guided by this discussion, two preliminary investigations were found to be necessary, including surveying the ground surface and locating the slip surface.

### 5.3.1 Ground Survey

Surveying the ground surface profile of each mudslide permits the accurate explanation of morphological characteristics and a quantitative comparison of site differences. Those parts of each mudslide most suitable for instrumentation can also be identified. Methods of slope profile survey are widely discussed (Pitty, 1966, 1967, 1968, 1969; Savigear, 1956, 1967; Young, 1964, 1971). For this study transects were positioned between the slope crest and toe, along the centre of the mudslide. An angle distance survey was conducted using the Abney Level technique, giving an accuracy of ± 0.5% (Young, 1972, 1974). Standard measured lengths of 3.0 m were used to minimise the influence of microtopography (Gerrard & Robinson, 1971) but to maintain sufficient resolution for the identification of important breaks of slope. A qualitative profile description was also recorded.

At Stair Hole the complexity of the mudslides necessitated a more detailed survey. A Hillger Watts One Second Theodolite and attached Electronic Distance Measurer were used to map the area. A network of five fixed points was established on stable ground around the perimeter of each mudslide system and used to map the mass movements. Results are presented as slope profiles (Figures 5.1 - 5.4) and a map of Stair Hole (Figure 5.5). Mudslide morphology has its own terminology (Selby, 1982) and descriptions of mudslides (Brunsden, 1984; Hutchinson & Bhandari, 1971; Prior & Stevens, 1971; Whalley <u>et al</u>., 1984) identify three distinct zones; a bowl shaped headward feeder, a central section or track and a toe slope lobe which is usually pear shaped, due to lateral spreading. The mudslide surface profiles were used to identify these divisions and their detailed characteristics (Table 5.2).

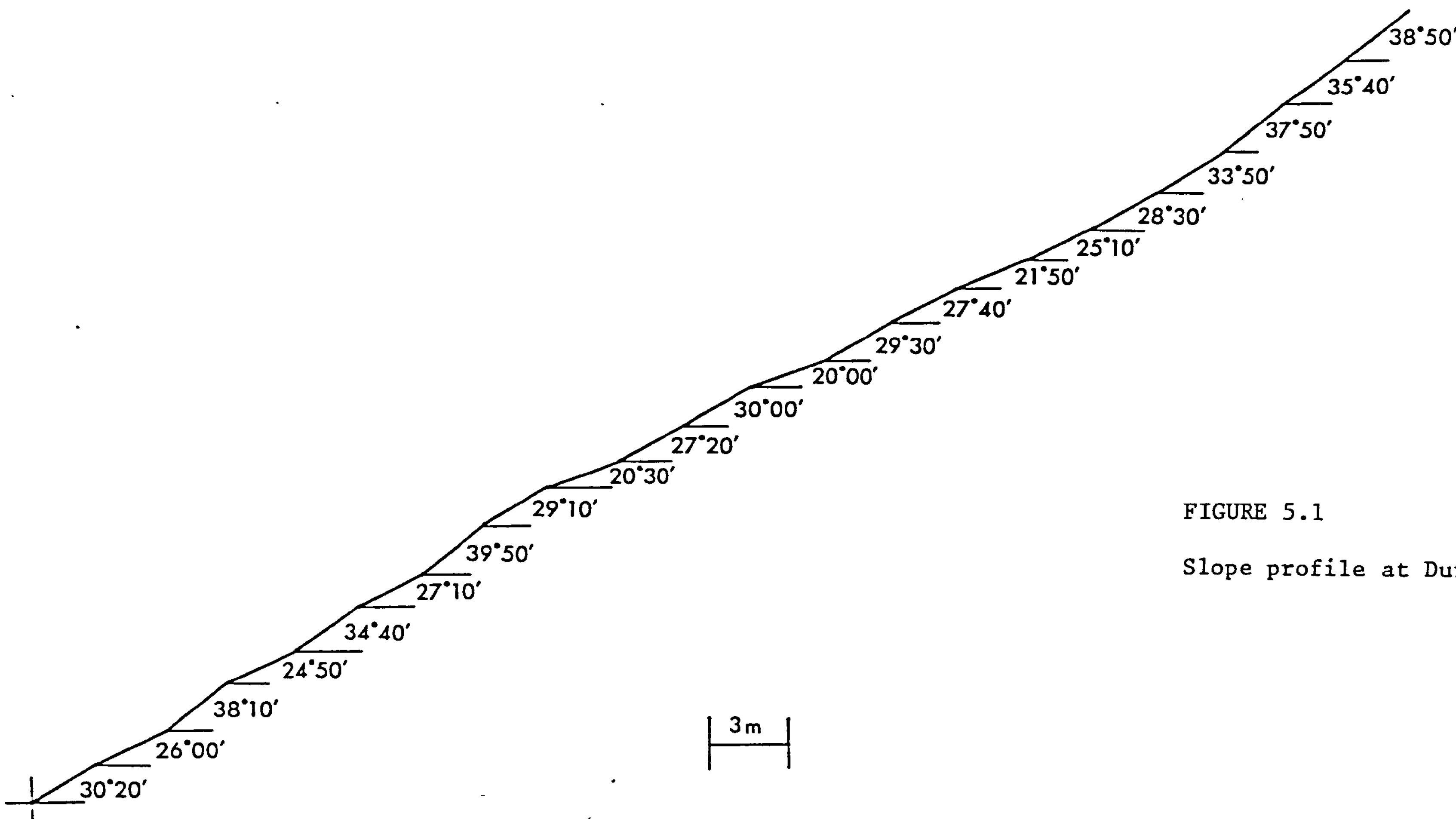



FIGURE 5.1

Slope profile at Durdle Door

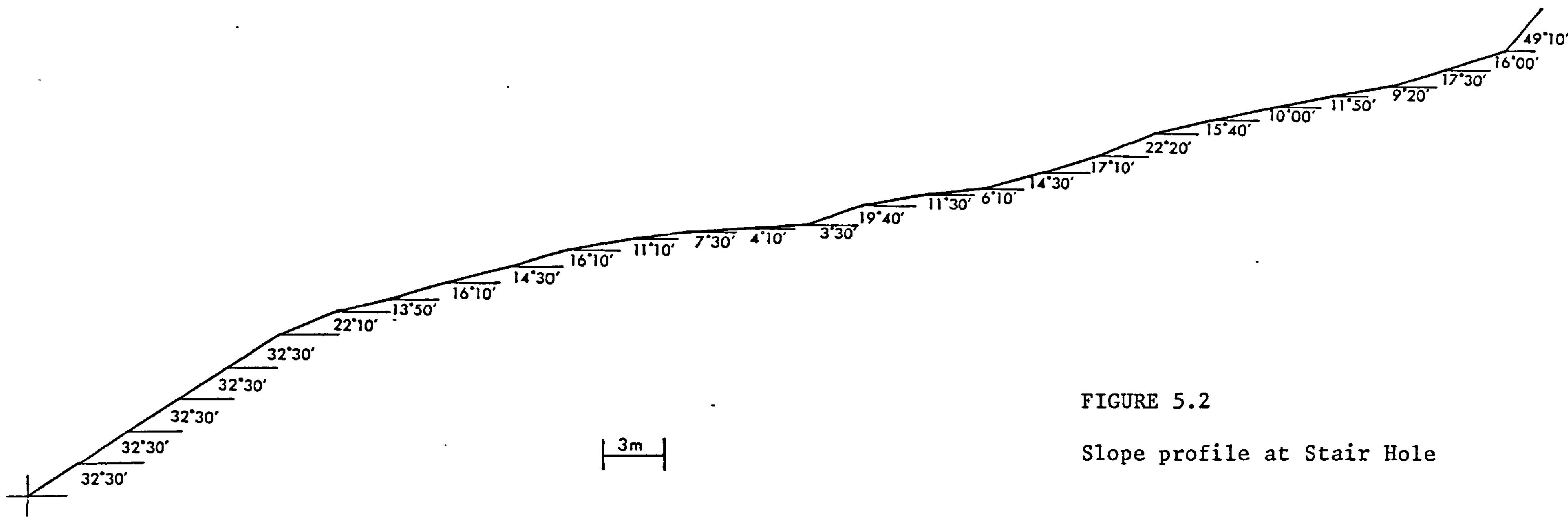



FIGURE 5.2

Slope profile at Stair Hole

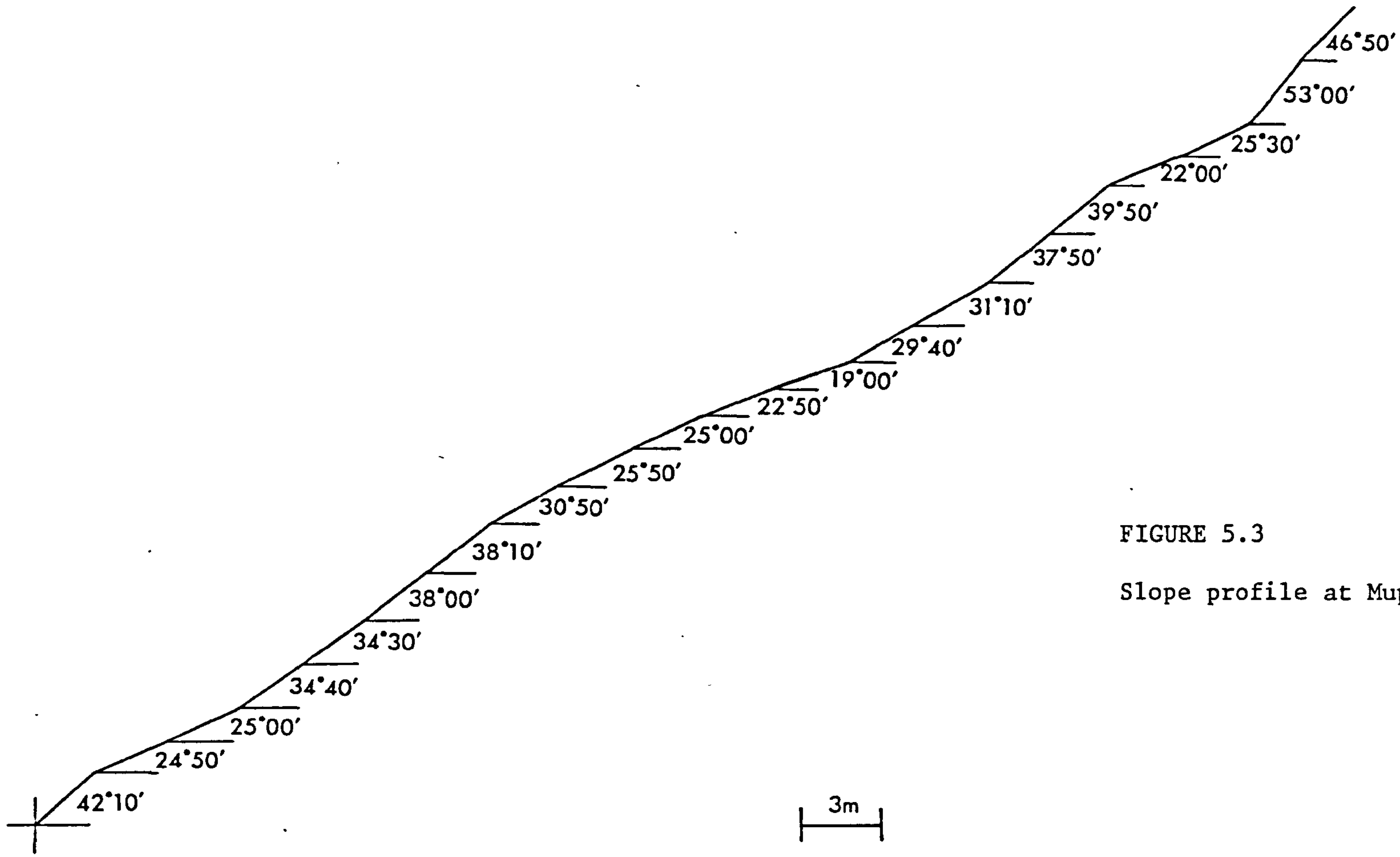



FIGURE 5.3

Slope profile at Mupe Bay

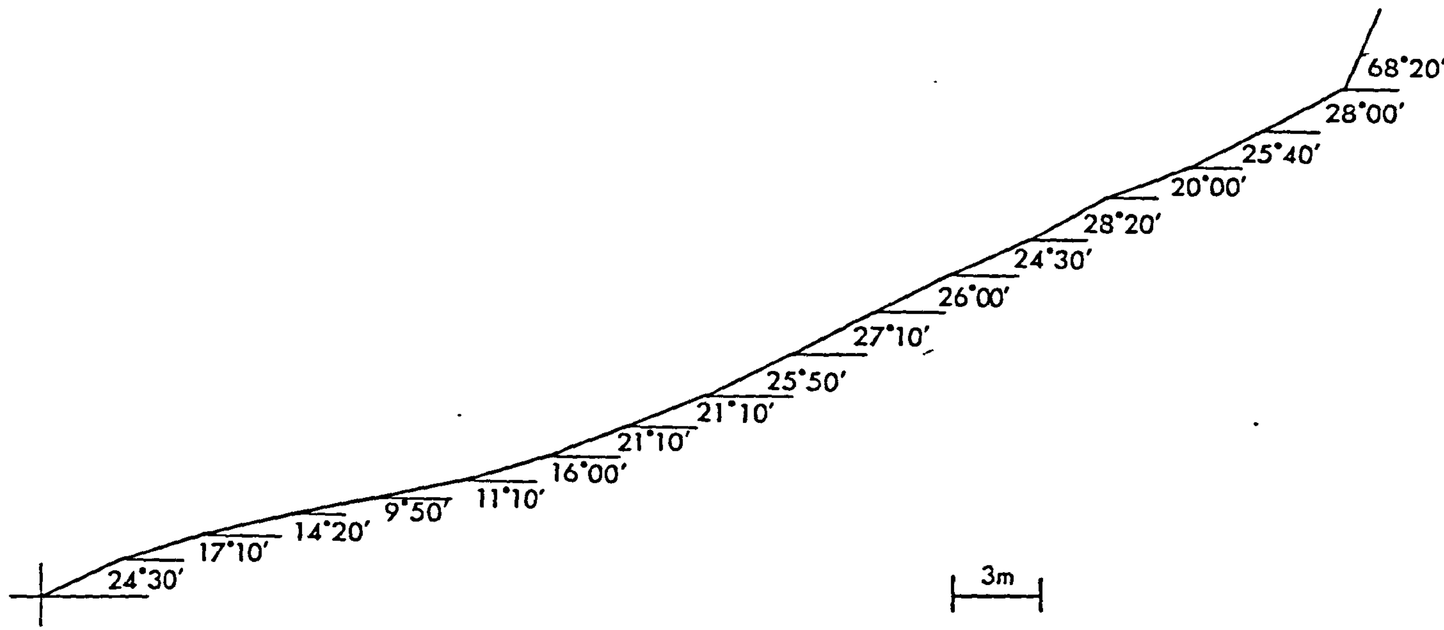



FIGURE 5.4

Slope profile at Worbarrow Bay

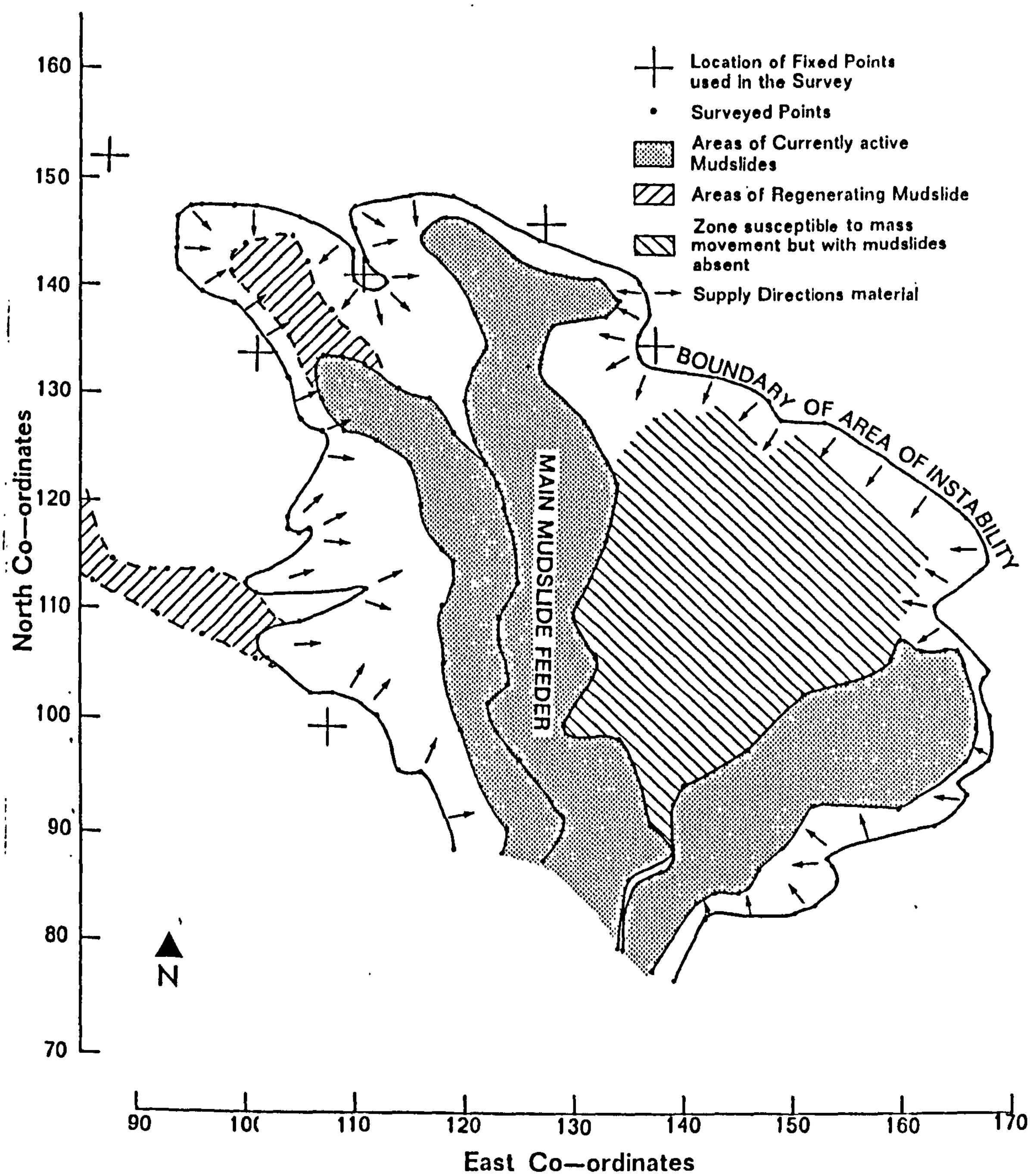



FIGURE 5.5

Survey results of the instrumented mudslide system at Stair Hole

TABLE 5.2

Characteristics of the mudslide surface identified from slope profile surveys

| Characteristics | Durdle Door | Stair Hole | Mupe Bay | Worbarrow Bay |
|---|---|---|---|---|
| Toe lobe characteristcs | Lobe at sea level spreading out over beach material. Rapidly comes to rest due to porewater dissipation through shingle. | Not significant to stability of overall slide . Material at sea level separated from foot of track by distinct break of slope. Secondary mudflows from main feeder. | Not significant to stability of overall slide. Lobe on beach is of material moving rapidly over steep lower slope section. | Lobe shows standard pear-shape characteristics but is moving out over beach material and is thus well drained and stable. |
| Details of track | Step-undulations representing incorporation of previously detached blocks. | Step undulations due to large incorporated blocks in central track section. | Relatively straight section of reasonably constant slope angle. Tension cracks etc show signs of rapid movements at differential rates up slide. | Straight track slightly convex in form with standard convex curvature to cross-section profile. |
| Headward bowl characteristics | Absent due to mudslides either side of Durdle Promontory. | Developing within hollow comprising rear scar and cemented sandstone beds. | Appear to be two due to differential rates of movement between rear of the most active part of the slide and upslope extent of unstable ground. | Characteristic headward bowl with supply of material from rear scar. |
| Characteristics of backscar | Absent | Well developed at rear of main slide. Currently subject to denudation by weathering and sheet wash. | Very small. 20-30 cm only due to headward bowl characteristics. | Well developed showing all standard characteristics and experiencing annual retreat. Supplies material to headward bowl thus loading rear of slope. |
| Overall profile shape | Approximately concave with marked basal concavity between toe lobe and track | Convexo/concave from slope toe to head due to steep slope section at foot of mudslide and rear scar. | Convexo-concave due to break of slope at slide toe and increasing mudslide surface angle towards slope rear. | Concave with marked concavities between toe lobe/end of track and bowl and rear scar. |

At Durdle Door (Figure 5.1) the track includes undulations, representing the detachment of intact blocks of material, which are currently moving downslope within the main mudslide mass. Mudslides occur on both sides of the Durdle Promontory and have all but coalesced at the head of the slope. Consequently there is no steep back wall at the rear of the track. Movement patterns were affected by this lack of headward loading (Hutchinson & Bhandari, 1971) due to the absence of a supply source at the slope crest. However, a number of undulations or ridges across the mudslide surface suggest previous temporal variations in the supply rate of material to the track.

The mudslides at Mupe Bay are found in depressions resulting from the removal of soft argillaceous material within the Wealden Beds, occurring between weathering resistant cemented sandstone units (chapter III). A marked steepening of slope angle occurs at the bottom of the track. During periods of instability, mobile material surges over this section and the slope angle is steep enough for the slip surface to become completely exposed. The downslope limit for instrumentation is clearly controlled by this morphology. The rear of this mudslide at Mupe Bay is also complex. Measurements suggest that the headward feeder divides into two sections. An area at the top of the slope comprises disrupted turf blocks which have become detached from the stable ground, displaying limited movement. Secondly, a bowl representing the rear of the currently most active part of the mudslide occurs part way downslope. The precise location of equipment at Mupe Bay was therefore restricted, despite the chosen slide being the most suitable for instrumentation.

Some of the stratigraphic and structural controls at Mupe Bay appear to be present at Worbarrow. The mudslide is moving in a well defined channel and the standard concave-up profile reported by other workers (Brunsden, 1984; Hutchinson, 1973; Hutchinson & Bhandari, 1971; Prior et al., 1968, 1971) is well developed, with a lobe spreading out over the beach beneath a straight track and steep rear scarp. In places, the mudslide surface is strewn with large sandstone boulders which have become detached from the rear of the slide and are now being rafted downslope on the main part of the matrix. The particularly uneven ground surface at the rear of the Worbarrow mudslide and the near vertical rear scar suggests that material regularly becomes detached, and loads the rear of the slide. The slope profile indicates, however, that blocks soon become broken down by weathering and incorporated within the main mudslide mass. Small mudflows feed on to the main slide from adjacent slope sections during particularly wet seasons. These are sufficiently large to cover installed equipment and cause disruptions to sampling from poorly sited apparatus. Finally, it appears that multiple lateral shears are present at Worbarrow Bay. The careful location of apparatus is therefore necessary, so that results can be considered a true record of mudslide movement.

The slope profile at Stair Hole reveals that the band of Jurassic strata causing a break of slope at the toe of the slide is important for two reasons. Firstly, it provides support to upslope material, although in other respects it prevents the accumulation and slope flattening usually provided by the toe lobe. Secondly, the Purbeck is a more effective barrier to marine attack than the Wealden. The direct effects of littoral processes as an erosive agent on the Wealden will therefore be minimised. Nevertheless, since the mudslide material overrides the barrier, the toe situation is one of partial to complete basal removal of debris, depending on the relative balance between the two processes. Secondary mudflows in the Wealden move over this surface from the toe of the mudslide track onto the beach. Stepped breaks of slope, identified by surveying the mudslide track, represent intact blocks of material previously detached from stable ground and now moving downslope as part of the main mudslide mass. Equipment location at Stair Hole was governed by a number of factors identified by the slope profile survey. As slope angle changed along the mudslide, so did the apparent moisture content. The siting of instruments accounted for this change. Due to the complex interactions of the different mudslides at Stair Hole, identified from the geomorphological mapping programme, a more detailed survey was undertaken to identify other factors important to the positioning of equipment. Survey results (Figure 5.5) suggested that precautions had to be taken to ensure that measurements at one mudslide at this location were representative of overall site conditions. There was also evidence that at some points, one mudslide moves over another, Care was necessary since surface detail alone did not necessarily establish the most suitable sampling points. Finally, as movement occurs, fissures develop across the mudslide. It was thought that these factors would significantly affect results. They were therefore taken into account in the interpretation of results. Finally, surveying revealed much evidence of human activity. This significantly influenced results where the ground was very soft. Equipment was therefore placed to minimise these errors. The slope profiling and survey therefore identified sources of potential error, which were minimised by the careful choice of instrumentation points. Precise equipment installation details will be discussed relative to individual techniques.

### 5.3.2 Slip Surface Identification

Locating the slip surface is a major prerequisite to detailed ground investigations (Hutchinson, 1982) and the reasons for doing so are numerous. The three-dimensional geometry of mudslides can be established and both volumes of moving material and a mass transfer budget calculated. The data is necessary for slope stability analysis and also to enable the correct installation of monitoring equipment. Slip surface depth was identified by the contrast in material properties above and below the boundary. This technique is widely used (Brunsden & Jones, 1976; Chandler, 1976; Early & Jordan, 1985; Hutchinson & Gorstelow, 1976; Skempton & Hutchinson, 1969; Skempton & Weeks, 1976).

Recordings were taken along transects using a 1.5" diameter screw thread auger. The results of geomorphological mapping and slope profiling were used to establish a suitable sample frequency. The contrast between the fabric of the slipped mass and the undisturbed _in situ_ strata is usually striking (Brunsden, 1984; Hutchinson, 1970) and this proved to be the case at the field sites. The depths at which these changes occurred were recorded. Three problems were encountered. Lumps of unweathered material within the argillaceous matrix occurred at some points. Augering was consequently required below material textural changes, to ensure that the measured distance represented the true depth to the slip surface. Multiple shears were recognised at some points. The one thought to exert greatest control on movement was identified. Finally at the rear of the mudslides at Mupe Bay and Durdle Door, the in situ characteristics of the matrix are well preserved and consequently the material contrast above and below the slip surface was difficult to identify. Results were corroborated by placing thin pine strips in the augered holes. These ruptured at the slip surface following movement. Various techniques based on this principle are reported (Cornforth et al., 1973; Eide & Holmberg, 1972; Hickl, 1977; Kennedy, 1972). For this study each hole was extended to a depth of 50 cm below the slip surface. Brittle pine strips 0.25 $mm^2$ were inserted into each hole and sharp sand was used to backfill the surrounding cavity. The strips were left in situ for a minimum three month period, subsequently removed and remeasured. Each set of results was cross-checked. Inhibiting problems included human interference with some of the installations at Stair Hole and the rupture of one piece at a shallow depth. This was thought to be due to poor installation.

At Stair Hole more detailed investigations were necessary. A geophysical survey was conducted using a Hammer Seismograph. Following Sowers & Royster (1978), this was used to supplement details from augering, with the former two techniques being used for calibration purposes. Augering permitted the location of the slip surface at specific points. Between each point interpolation was required to define the remainder of the boundary. Using a seismograph,complete transects were located and augered holes used to confirm the identified slip surface. This technique is becoming increasingly used in landslide studies (Hutchinson, 1982) and there are many examples of investigations of this type (Bogoslovsky & Ogilvy, 1977; Carroll et al., 1972; Cratchley, 1976; Tamaki & Ohba, 1971). The seismic refraction technique was adopted. Velocity nomograms were produced for a number of transects and a geomorphological cross-section was drawn for each one, to relate surface form and slip surface morphology. To ensure satisfactory recording, an inertia switch, attached to the shaft of the sledge hammer used to propagate the shock waves, provided a suitable means of triggering the seismograph. The time between propagation of the seismic wave at the hammer point and its first arrival at the geophones was recorded (Figure 5.6) and from these results a two-dimensional plot of the slip surface was obtained. Five criteria were used to locate seismic cross-sections. Details were required for further instrumentation. Results had to coincide with data from augering and the ruptured pine strips, permitting the cross-checking of results. Long-profile seismic lines were located along distance-angle transects. Finally, cross-sections had to be sufficient in number and suitably located to identify long-profile variations.

A number of general slip surface characteristics were noted (Table 5.3) as well as obtaining detailed sections (Figure 5.7 - 5.10). The pine strips at Durdle Door remained intact at three points. One of two possible reasons account for this, including particularly small movements during the period of installation and inadequate backfilling of those holes where the strips did not rupture. The slip surface at Durdle Door is particularly shallow, with no major depth variations in either cross-section or long profile. The results at Worbarrow Bay and Mupe Bay are similar. There is a clear difference at both these sites between the argillaceous mudslide matrix and the in-situ material. At these locations variations in the depth of the slip surface are due to surface

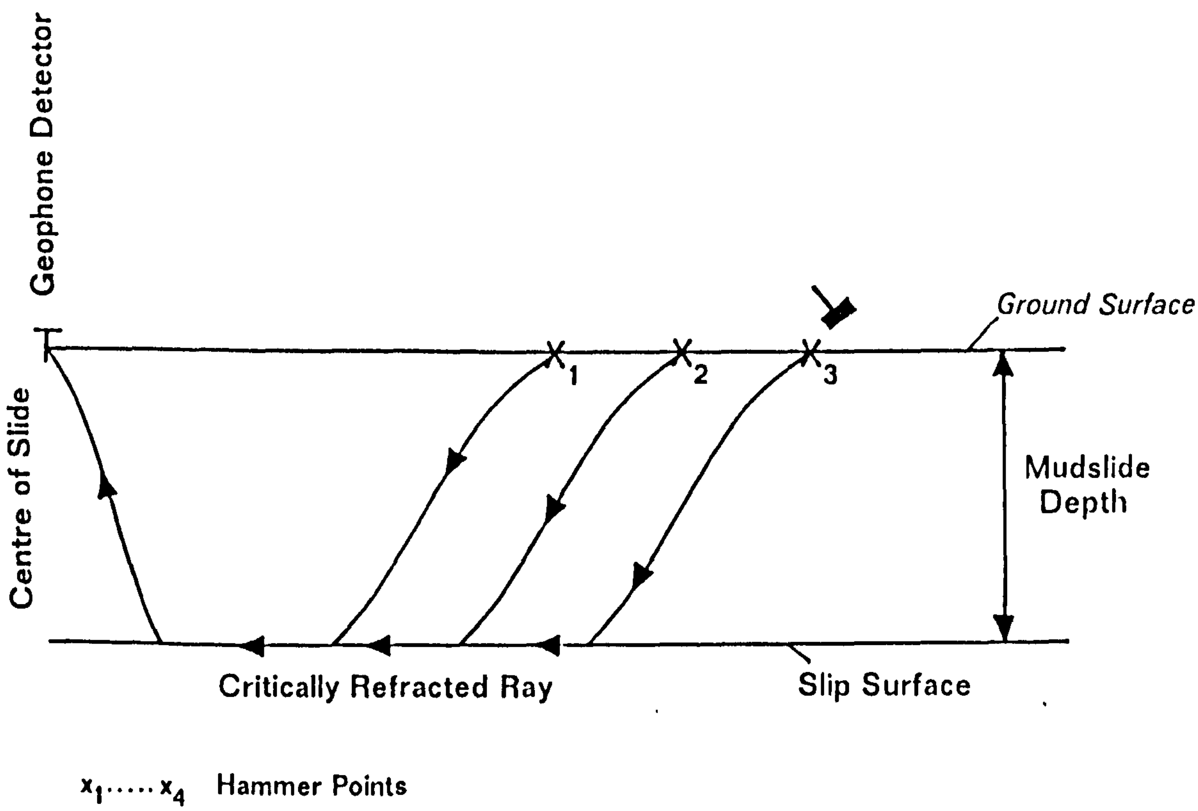



FIGURE 5.6

Geophone - plate spread used in conjunction with the Hammer Seismigraph

TABLE 5.3

Characteristics of mudslide slip surfaces identified from slope profile surveys

| Characteristic | Durdle Door | Stair Hole | Mupe Bay | Worbarrow Bay |
|---|---|---|---|---|
| Depth of slip surface | Relatively shallow feature with slip surface depth remaining relatively constant throughout length. | Relatively shallow feature. Varying depth throughout length. | Relatively shallow feature Thinning towards centre of slide but slip surface generally parallel to ground surface. | Shallow but greater depth than other measured sites Generally conforms to standard characteristics. |
| Long profile details | Concave, particularly at toe of slide as lobe moves over beach material. | Convexo-concave due to in-situ Purbeck at toe of slide. | Convexo-concave due to sharp break of slope at bottom of track. | Concave with 'characteristic' mudslide long profile. |
| Cross-section details | Elongated 'U' dropping quickly on both sides to relatively horizontal surface. | 'U' shaped. Some variation throughout length relative to width of slide. | Elongated 'U'. Flat bottomed. | 'U' shaped, increasingly elongated towards rear of slide as feature decreases in depth. |
| Lateral shears | Indistinct, defined most clearly due to presence of rills along shear in wet conditions. | Distinct due to adjacent mudslides moving seawards. Clear reidel shears towards toe of slide. | Easily identified lateral shears particularly towards toe of slide. Surface covering of sand washed from side of mudslide. | Clear lateral shears showing protrubing polished surface on in-situ material following rapid movement. Distinct reidel shears. |
| Other details | Shear position partly due to in-situ stratigraphy | | Steep sections of slope displaying exposed surface in very wet conditions | |

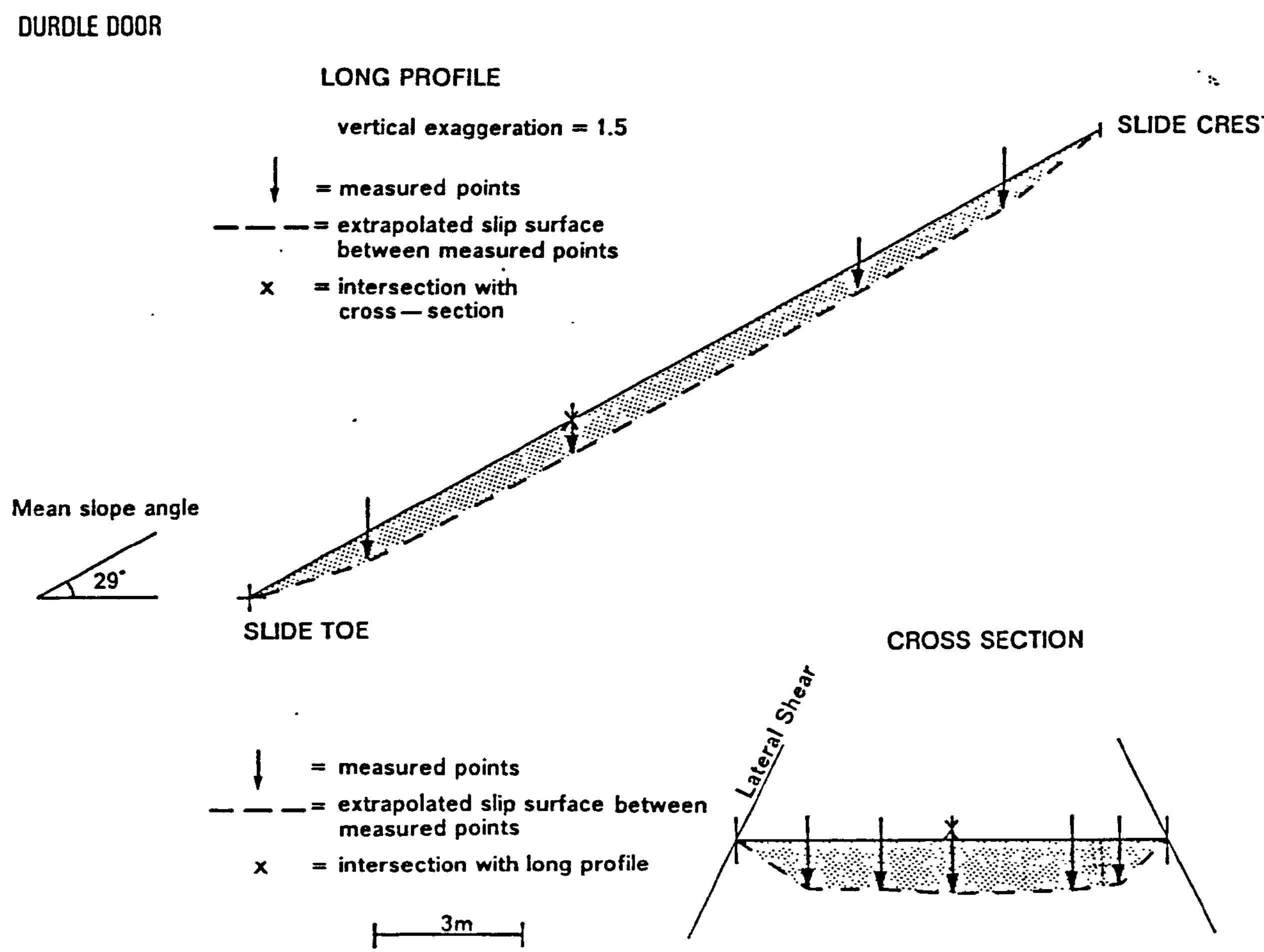



FIGURE 5.7

Slip surface profile of the instrumented mudslide at Durdle Door

MUPE BAY

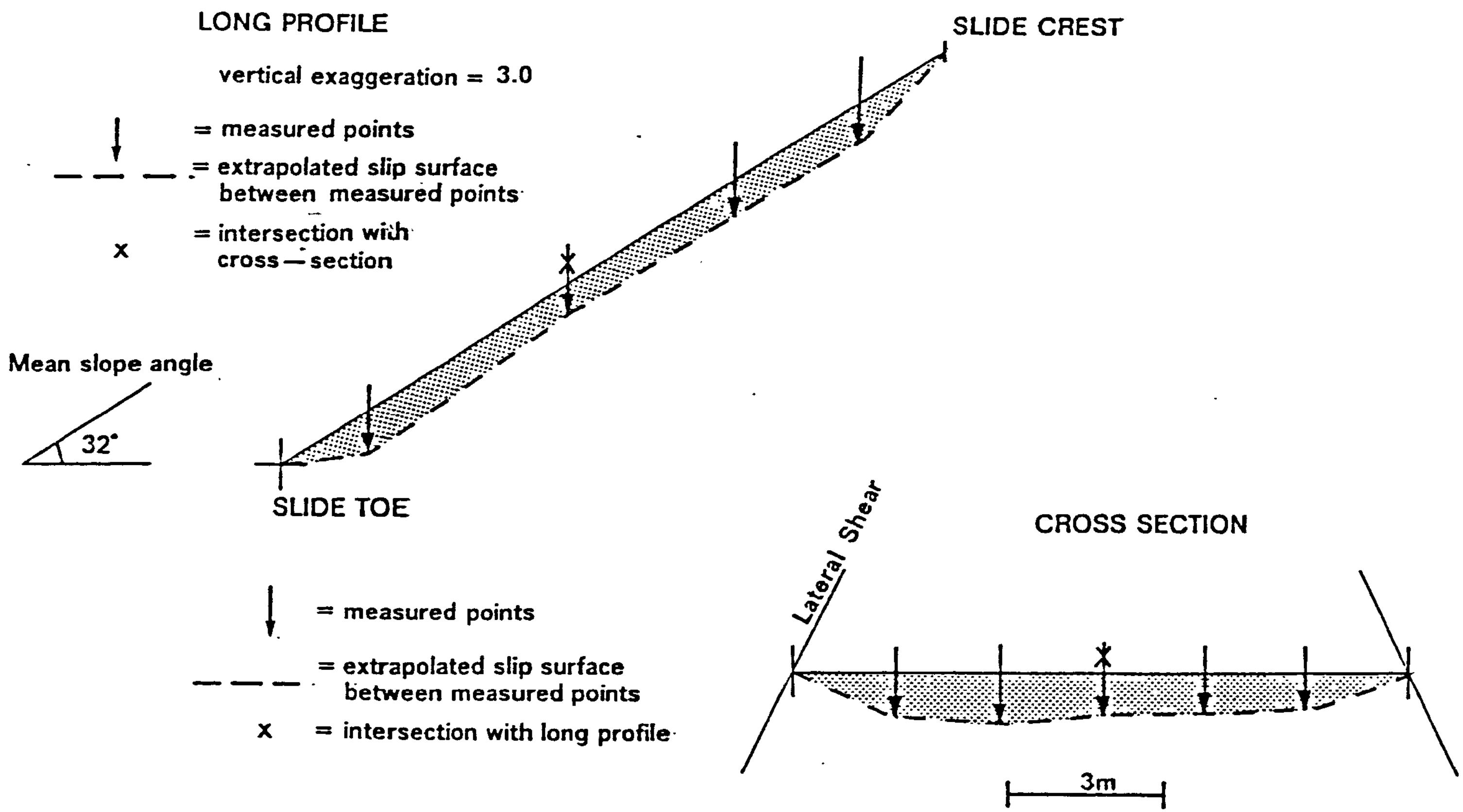



FIGURE 5.8

Slip surface of the instrumented mudslide at Mupe Bay

WORBARROW BAY

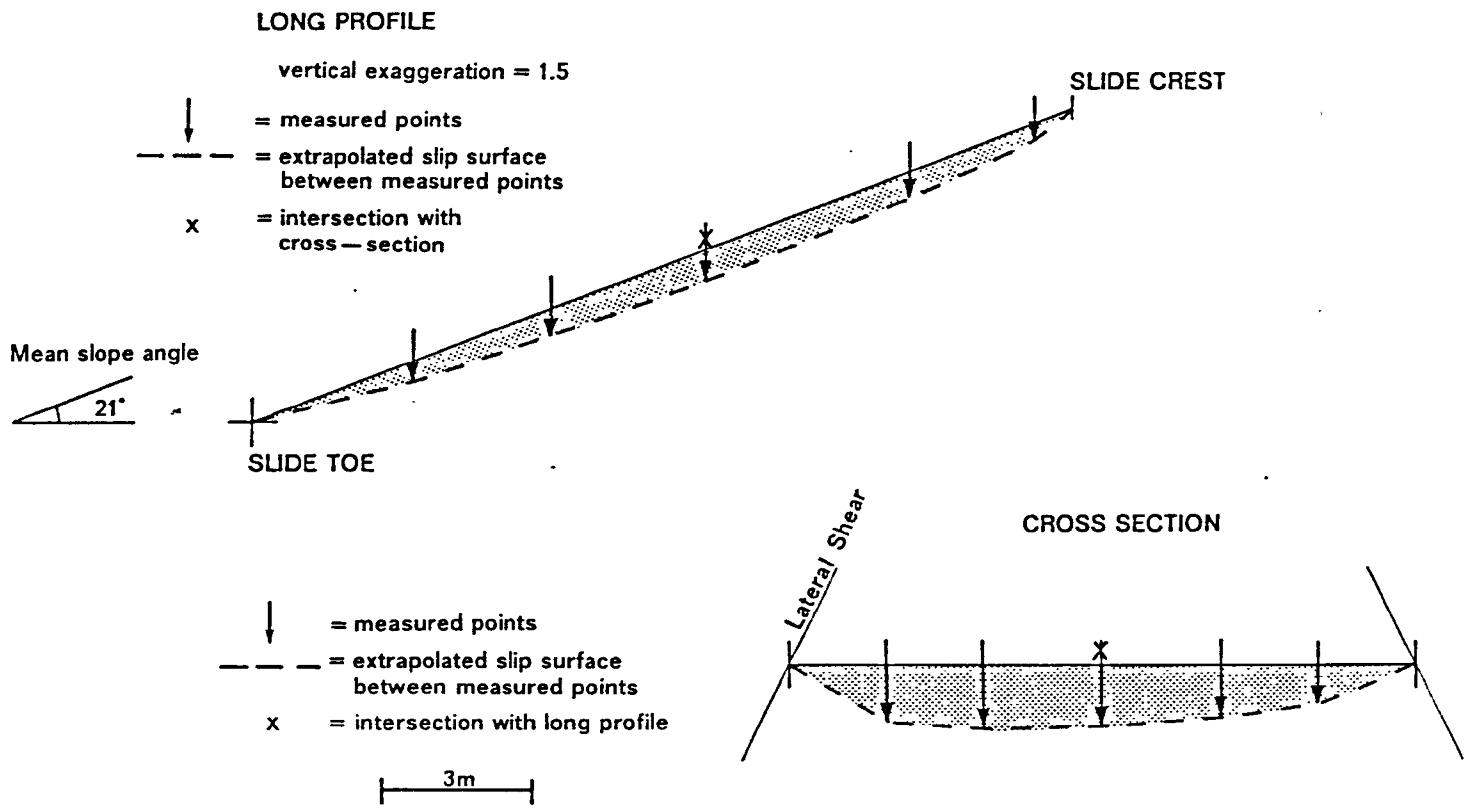



FIGURE 5.9

Slip surface profile of the instrumented mudslide at Worbarrow Bay

STAIR HOLE

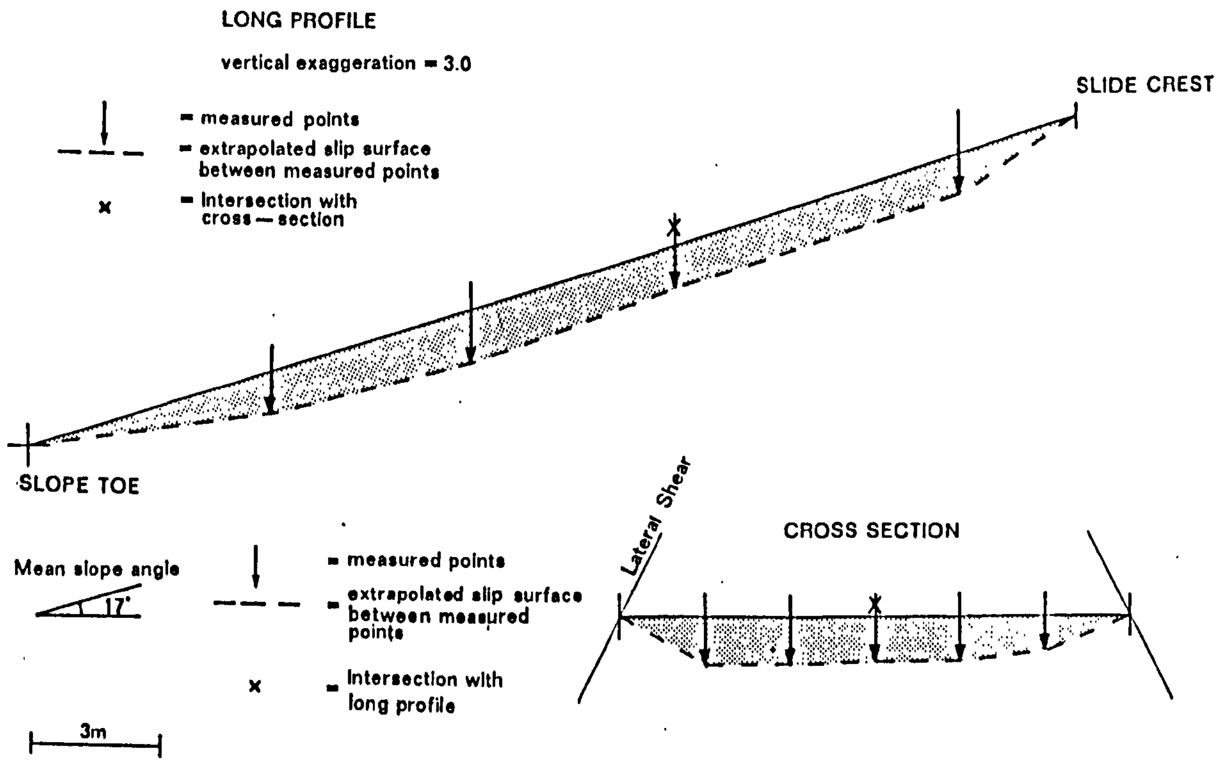



FIGURE 5.10

Slip surface profile of the instrumented mudslide at Stair Hole characteristics, including differences in the thickness of material across the surface due to differential supply rates, and a thinning of material where slope angle is sufficiently steep to increase movement rates. The profiles at Mupe and Worbarrow are similar to the characteristic forms described by Hutchinson (1970) and Hutchinson & Bhandari (1971). It is possible to calculate the volume of each mudslide from the results (Table 5.4) and by combining this data with details of movement, material mass transfer through the mudslide system can be established.

The slip surface profiles at Stair Hole identified using the Hammer Seismograph (Figure 5.11) are complex (Figure 5.12) but nevertheless self explanatory. Good correlations exist between these results and the location of the slip surface from the other utilised techniques. Each velocity nomogram presents a number of reflectors. The line on the chart representing the shortest travel time indicated the depth of the slip surface. Other reflectors were thought to be from geological characteristics, including stratigraphic differences and structural discontinuities at shallow depth. It became difficult to interpret results towards the edge of the mudslides between the points where the lateral shear daylights and the slip surface begins to curve upwards towards the ground. Results show that all cross-sections have flat slip surfaces below the main part of the mudslide mass, with variations in depth reflecting localised changes in ground surface topography, rather than undulations across the base of the slide. From these results, the total volume of this system can be established (Table 5.5) and used in conjunction with surface velocity measurements to establish a mass transport budget.

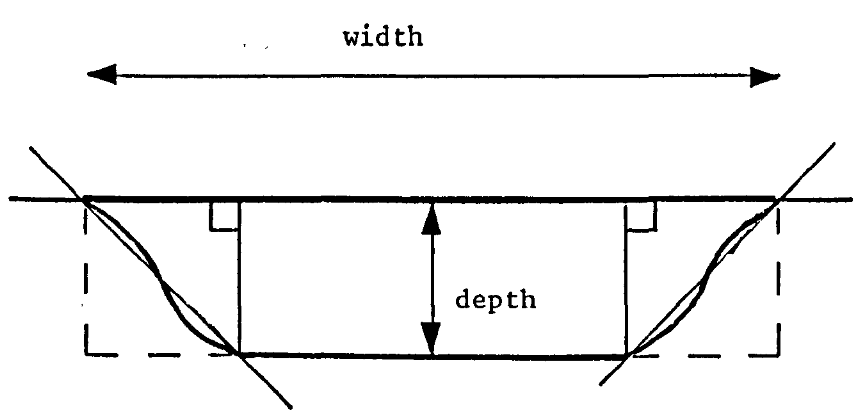
TABLE 5.4

Approximate mudslide volumes: Durdle Door, Mupe and Worbarrow

| Parameters | | Durdle Door | Mupe Bay | Worbarrow Bay |
|---|---|---|---|---|
| mean slide length (m) | | 63 | 60 | 51 |
| mean depth | toe slope section (m) | 1.34 | 0.79 | 0.91 |
| | mid slope section (m) | 1.18 | 0.51 | 1.27 |
| | top slope section (m) | 1.08 | 0.66 | 0.91 |
| mean width (m) | | 9 | 12 | 13 |
| mean x-section | toe slope ($m^2$) | 10.72 | 8.69 | 10.92 |
| | mid slope ($m^2$) | 9.44 | 5.61 | 15.24 |
| | top slope ($m^2$) | 8.64 | 7.26 | 10.92 |
| Approximate mudslide volume ($m^3$) | | 604.8 | 417.45 | 651.96 |

Notes:

Following assumptions are made:



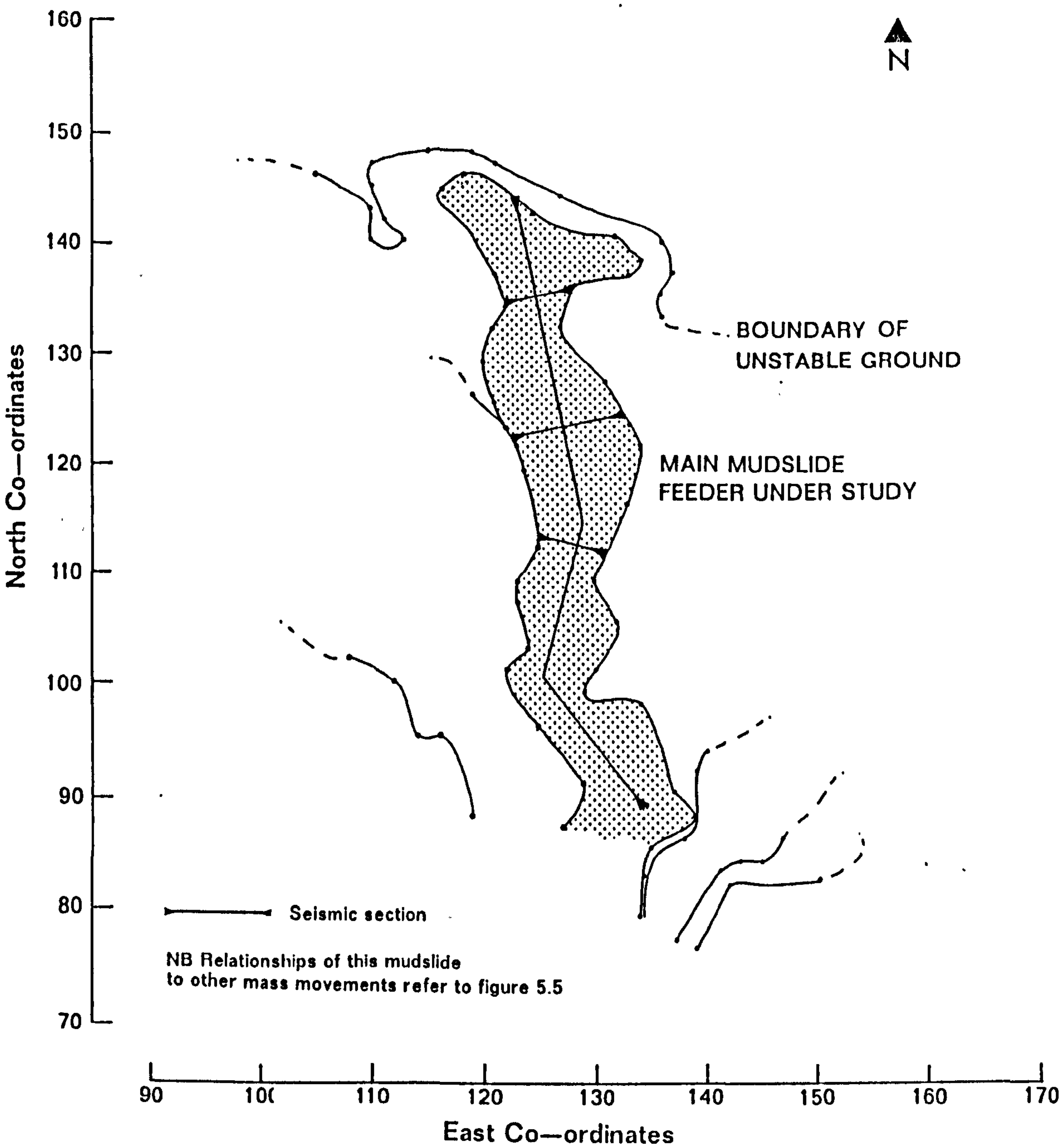



FIGURE 5.11

The location of sections investigated using the Hammer Seismograph at Stair Hole

NOTE: For relationships of this mudslide to adjacent mass movements refer to Figure 5.5

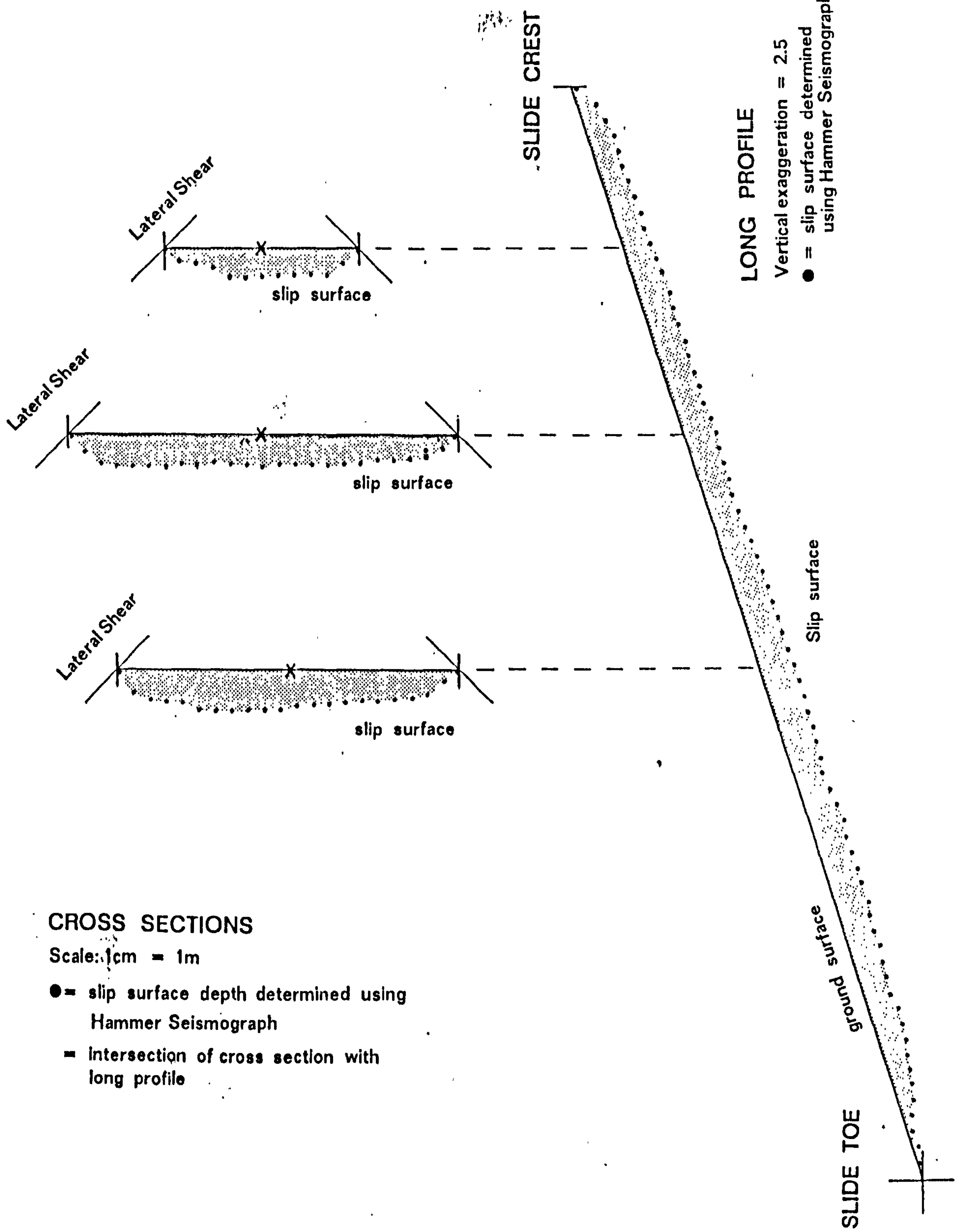



FIGURE 5.12

Slip surface profile at Stair Hole identified using the Hammer Seismograph

TABLE 5.5

Approximate mudslide volume Stair Hole

| Parameters | Unit Measurements | Individual Slide Volumes | Cumulative Volume |
|---|---|---|---|
| MAIN ACCUMULATION FEEDER | | | |
| slide length (m) | 81 | | |
| mean depth (m) ( toe slope | 1.03 | | |
| ( lower slope | 1.27 | | |
| ( upper slope | 1.03 | | |
| ( top slope | 0.91 | | |
| mean width (m) ( toe slope | 16 | | |
| ( lower slope | 22 | | |
| ( upper slope | 24.8 | | |
| ( top slope | 25 | | |
| mean x-section ($m^2$) ( toe slope | 16.48 | | |
| ( lower slope | 27.94 | | |
| ( upper slope | 25.54 | | |
| ( top slope | 22.75 | | |
| Approximate volume ($m^3$) | | 1864 | 1864 |
| N.W. SIDE FEEDER | | | |
| slide length (m) | 25 | | |
| slide width (m) | 10 | | |
| depth to slip surface (m) | 0.73 | | |
| approx volume ($m^3$) | | 182.5 | 2046 |
| S.W. SIDE FEEDER | | | |
| slide length (m) | 27.2 | | |
| slide width (m) | 12.2 | | |
| depth to slip surface (m) | 0.52 | | |
| approx volume ($m^3$) | | 172.5 | 2219 |
| N.E. SIDE FEEDER | | | |
| slide length (m) | 29.6 | | |
| slide width (m) | 10.2 | | |
| depth to slip surface (m) | 0.64 | | |
| approx volume ($m^3$) | | 193.23 | 2412 |
| S.E.SIDE FEEDER | | | |
| slide length (m) | 28 | | |
| slide width (m) | 12.2 | | |
| depth to slip surface (m) | 0.58 | | |
| approx volume ($m^3$) | | 198.13 | |
| VOLUME OF MUDSLIDE SYSTEM (m3) | | | 2610 |

These investigations were essential prerequisites to the ground investigation programme, providing an accurate means of locating equipment and permitting the detailed consideration of variable interrelationships, site characteristics and slope stability.

## 5.4 FIELD INSTRUMENTATION

The initiation of movement in mudslides is due to the effects of variables such as the water budget, porewater pressure and material supply (Brunsden, 1984). Field instrumentation for monitoring such parameters is widely reported (American Soc. Civil Engineers, 1972; British Geotechnical Society, 1974; Franklin, 1984; Goudie, 1981; Wilson & Mikkelsen, 1978) although it is often emphasised, by Peck (1972) for example, that instrumentation does not replace field observations and preliminary investigative procedures. The field instrumentation programme adopted for this study is best outlined by considering the mudslide as a process-response system (Brunsden, 1973) (Figure 5.13). The parameters requiring measurement include rate of input, porewater pressure, surface movement, subsurface movement and climate. Each variable was monitored at two scales using different techniques, one involving standard equipment which was installed at all four sites and the other utilising electronic data gathering apparatus. In some instances this involved the design and construction of new apparatus. Due to high costs,these instruments were installed at one site only.

### 5.4.1 Rate of Input

Inputs to the mudslides include material from the rear scar, the side walls and the base of the flow. The supply from the back scar is usually measured either by erosion pins (Brunsden, 1973) or surveying (Hutchinson, 1970) depending on the rate of addition. Visitor pressure is too great at Stair Hole and Mupe Bay to permit the use of these techniques, while at Durdle Door geomorphological mapping shows this part of the instrumentation not to be necessary due to the absence of a rear scar. A pilot study was conducted at Stair Hole. Of 20 pins installed across the rear scar, 14 had been removed and the remainder interfered with within a fortnight. The following technique (Figure 5.14) was therefore utilised. Dowel pieces 2 cm in diameter were cut to 10 cm lengths, pointed at one end and a 2.5 cm flat-head nail hammered into the opposite end. Five pegs were installed flush with the ground surface, in a straight line immediately behind and at 90$^{o}$ to the rear scar. Each was separated by 20 cm. For remeasurement each peg was located with a metal detector and the distance between the crest of the rear scar and the closest pin measured. Only a general estimate of supply was possible by this technique and despite attempts to conceal the pegs some were removed in the middle of the sampling programme.

Detailed monitoring at Worbarrow Bay was possible due to restricted public site access. Sequentially numbered mudguard washers were each threaded with a 6" nail, a 2 m square grid marked out on the rear scar of the monitored slide and a nail hammered into each intersection on the trellis (Figure 5.15) (Plate 5.1). In the central section of the rear scar an additional network of 1 m long brass rods was

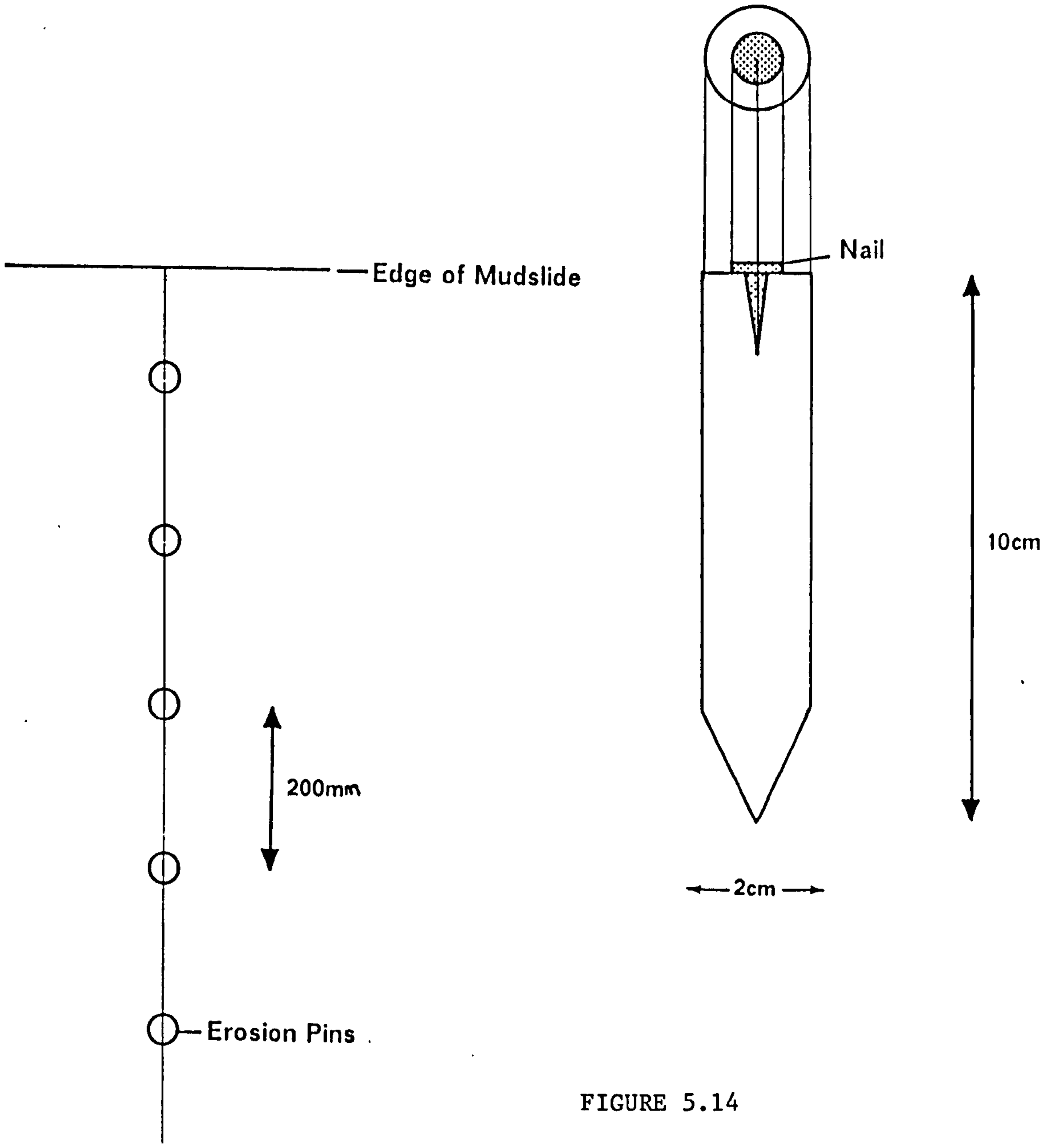



FIGURE 5.14

Erosion pin installation at each site to estimate mudslide rear scar input

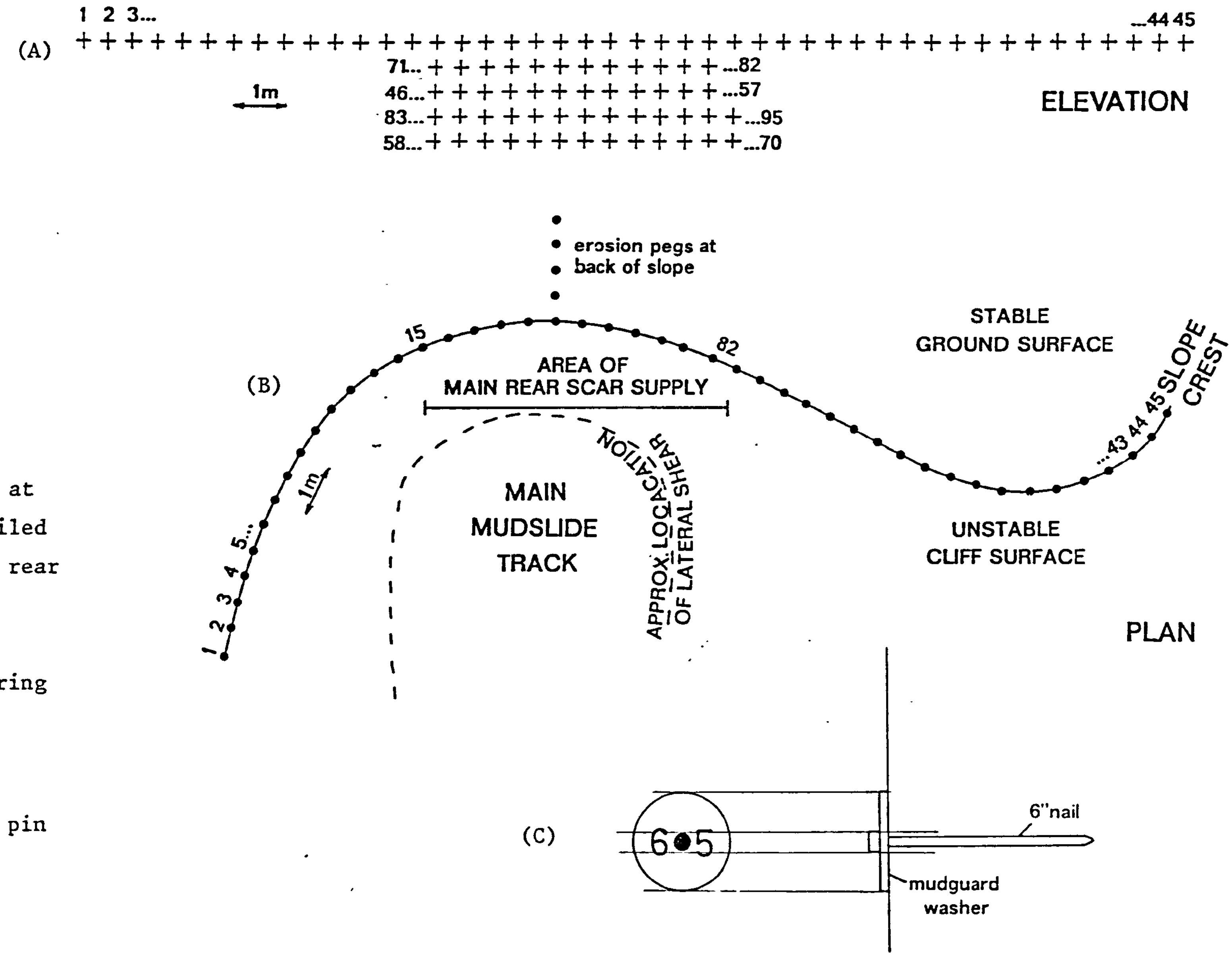



FIGURE 5.15

Erosion pins installed at Worbarrow Bay for detailed estimation of mudslide rear scar input

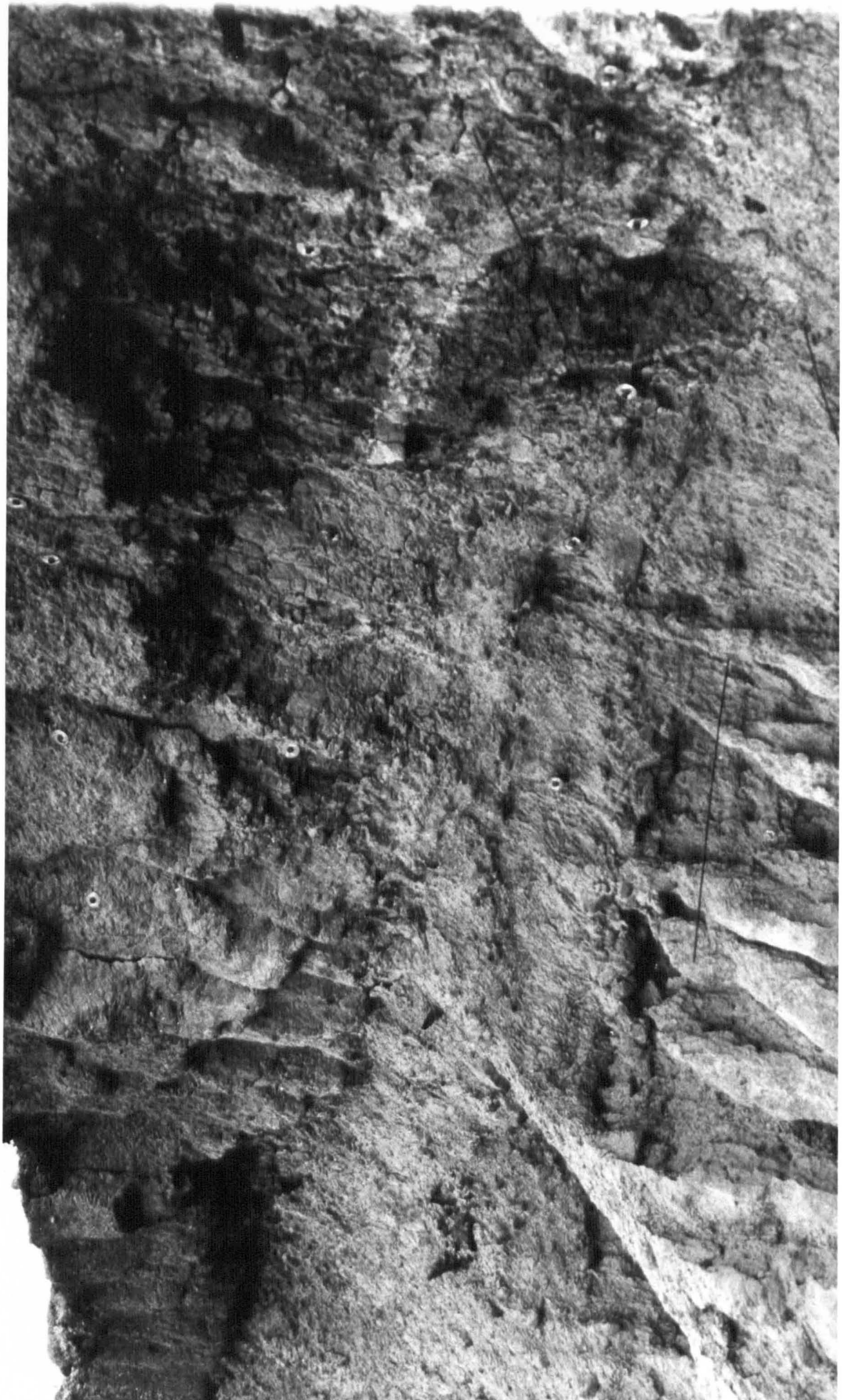
(A) Installation numbering sequence

(B) Outline plan

(C) Details of erosion pin

PLATE 5.1

Erosion pins installed at Worbarrow Bay



installed. The effects of surface weathering were calculated by using a modified set of callipers to measure the distance between the mudguard washer and the scar surface. Completely detached blocks were identified from the pins, which usually remained attached following failure. Each block was measured, the pin removed and replaced in the freshly exposed face. Occasionally blocks became detached from points between the pins, but this debris input appeared to be small. Also, disintegration of large blocks on detachment occasionally made it difficult to directly calculate the amount of loading and the dimensions of the fresh scar had to be used as a surrogate measurement. Supply of material in significant amounts was infrequent and when it did occur, it was easily identified. Individual events were noted and the approximate volume of each one was calculated.

Base erosion is usually measured by excavating a trench and inserting erosion pins across the slip surface (Brunsden, 1973). It was concluded that the disruption to the mudslide necessary for the insertion of these pins would probably have a greater effect on the overall results than the monitoring programme itself.

### 5.4.2 Porewater Pressure

Not only is an understanding of the distribution of pore pressures important to any slope stability study (Carson & Kirkby, 1972) but it is recognised that fluctuation in groundwater is the most important expression of climate (Bhandari & Hutchinson, 1984; Hutchinson, 1970, 1974; Hutchinson & Bhandari, 1971; Hutchinson et al., 1974; Whalley et al., 1984). A wide variety of measuring devices are available (Table 5.6) and their use is widely reported (Anderson & Kneale, 1980; Bishop et al., 1964, 1969; Hutchinson, 1970; Vaughan, 1973). Many technical papers discussing this specific variable are also available (Bhandari & Hutchinson, 1981; Brunsden, 1973; Craig, 1979, 1981; Gibson, 1963; Hutchinson, 1973; Hutchinson & Bhandari, 1971; Hutchinson et al., 1974; Penman, 1960; Prior, 1973; Prior & Stevens, 1972; Vaughan, 1969, for example). At each site monitoring points were located between the toe and the rear scar. Holes were excavated with a 6" diameter bucket auger to a point just above the slip surface and Casagrande-type standpipe piezometers installed. A porous tip 19 mm in diameter, 300 mm long, pore diameter of 60 microns and permeability of $3 \times 10^{-4}$ $ms^{-1}$ was chosen. The piezometer tip was surrounded with sharp sand to act as a filter and a bentonite-clay mixture was used to form a plug (Figure 5.16). The remaining cavity was backfilled with previously extracted material.

For protection against vandalism the piezometer access tubes were cut off just below the ground surface, sealed with an end cap, covered by a metal plate and the ground surface re-turfed. Sampling required relocation of equipment with a metal detector, uncovering the tube and measuring the water level with a dipmeter fitted with an 11 mm diameter PTFE brass probe. Despite these precautions a number of installations were interfered with and sampling had to be curtailed at these points.

Detailed monitoring was conducted at Worbarrow Bay. Two water depth sensor, highly sensitive steel diaphragm pressure transducers (Table 5.7) (Figure 5.17) were installed in boreholes in a similar manner to the standpipe piezometers with one at the toe and the

TABLE 5.6

Equipment previously used for measuring pore water pressure

| Report of Technique | Date | Location of Study | Technique Used |
|---|---|---|---|
| Chandler | 1971 | Rockingham, Northants | Casagrande standpipe piezometer |
| Craig | 1981 | E Co Antrim, N Ireland | Casagrande standpipe piezometer |
| Brunsden | 1973 | Stonebarrow, Dorset | Casagrande standpipe piezometer |
| Hutchinson | 1970 | Beltinge, Kent | Casagrande standpipe piezometer |
| Skempton & Hutchinson | 1970 | Beltinge, Kent | Casagrande standpipe piezometer |
| Anderson & Kneale | 1980 | M6 Motorway Embankment | Tensiometers |
| Hutchinson & Bhandari | 1971 | Isle of Wight | Electrical diaphragm piezometers |
| Prior et al | 1971 | Minnis North, N Ireland | Electrical diaphragm piezometers & Peekel micro strain guage |
| Burt | 1978 | Quantock Hill, England | Automatic fluid-scanning switch tensiometer system |
| Pethick | pers comm | Holderness, N. Humberside | Automatic fluid-scanning switch tensiometer system |

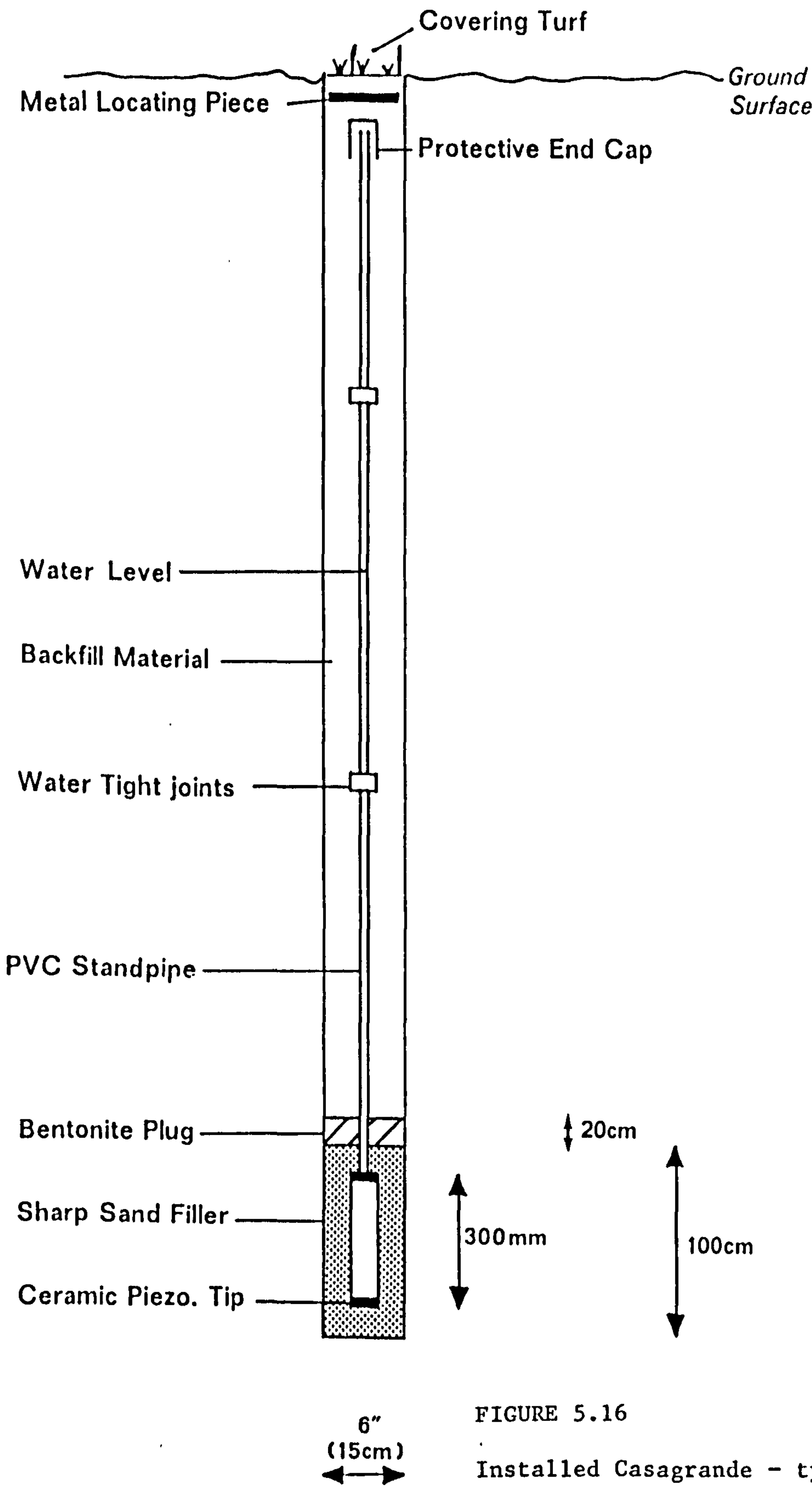



FIGURE 5.16

Installed Casagrande - type standpipe piezometers

TABLE 5.7

Pore water pressure transducer technical details

| Technical details | Sensor 1A-845-A | Sensor 1A-846-A |
|---|---|---|
| Range (Bar G) | 0-1 | 0-1 |
| Supply (Volts A.C.) | +10.0 | +10.0 |
| Sensitivity ($mv^{1}v^{-1}BARG^{1}$) | 5.88 | 5.84 |
| Non Linearity & Hysteresis (% F.S.) | $\pm$0.02 | $\pm$0.10 |
| Full Range Output (mV) | 58.8 | 58.4 |
| Input Resistance | 1761 | 1726 |
| Output Resistance | 434 | 455 |

Manufacturer: Shape Instruments Ltd

Transducer Type: Water Depth Sensor

Type Number: SH2-P-5300-SP65L-15MO

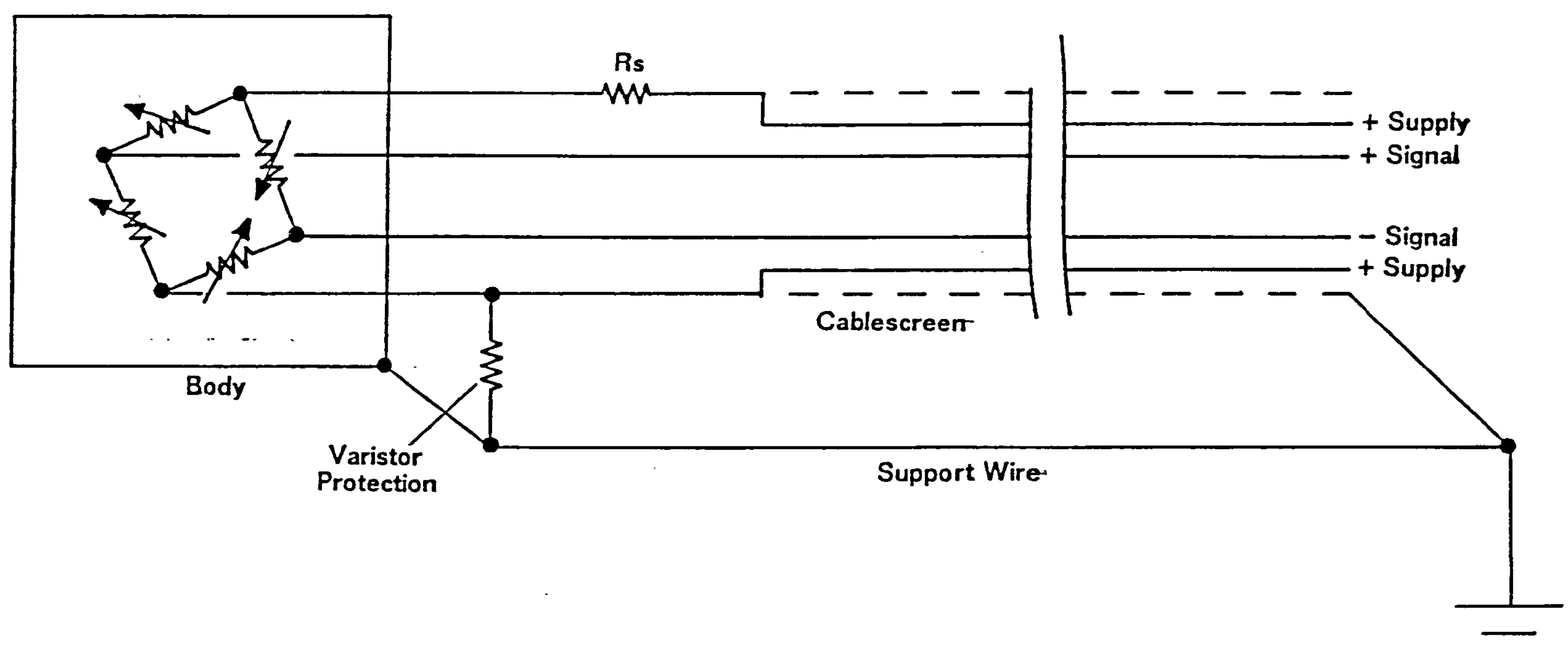



FIGURE 5.17

Details of the 'Shape' pressure transducers used for monitoring pore water pressure other towards the top of the mudslide. Each had been previously calibrated in the laboratory. Results were logged on an Epsylon Data Logger via an analogue: digital interface and tapes were translated using a PDP8 computer to give details of the head of water. Numerous problems initially beset this equipment and the system had to be repeatedly removed, modified, recalibrated and reinstalled. These problems resulted in loss of data. The energising batteries completely lost their charge within seven days on two occasions, despite being fully charged for field installation. Malfunctioning of the tape advance mechanism due to motor faults made some weekly runs of data impossible to translate. The length of the cable connecting the transducer and Epsylon Logger and coiling the cable affected transducer calibration. These errors were only found following field instrumentation. Problems were also encountered due to drift in the results as the energising batteries discharged. This was overcome by designing an interface unit for the energising battery/transducer link. The initial circuit had to be redesigned on two occasions before satisfactory results were obtained (Figure 5.18). The pressure transducers were calibrated in the laboratory using standpipe data, but initially no results were obtained. Recalibration to zero head of water gave satisfactory results. This has general implications for the measurement of porewater pressure on unstable ground. Mudslides frequently have shears and tension cracks crossing their surface which can extend well down into the argillaceous material. Recent groundwater research (Bell, pers. comm.) suggests that the presence of microfissures around wells and standpipe piezometers can greatly affect results. It therefore not only seems likely that the results from standpipe

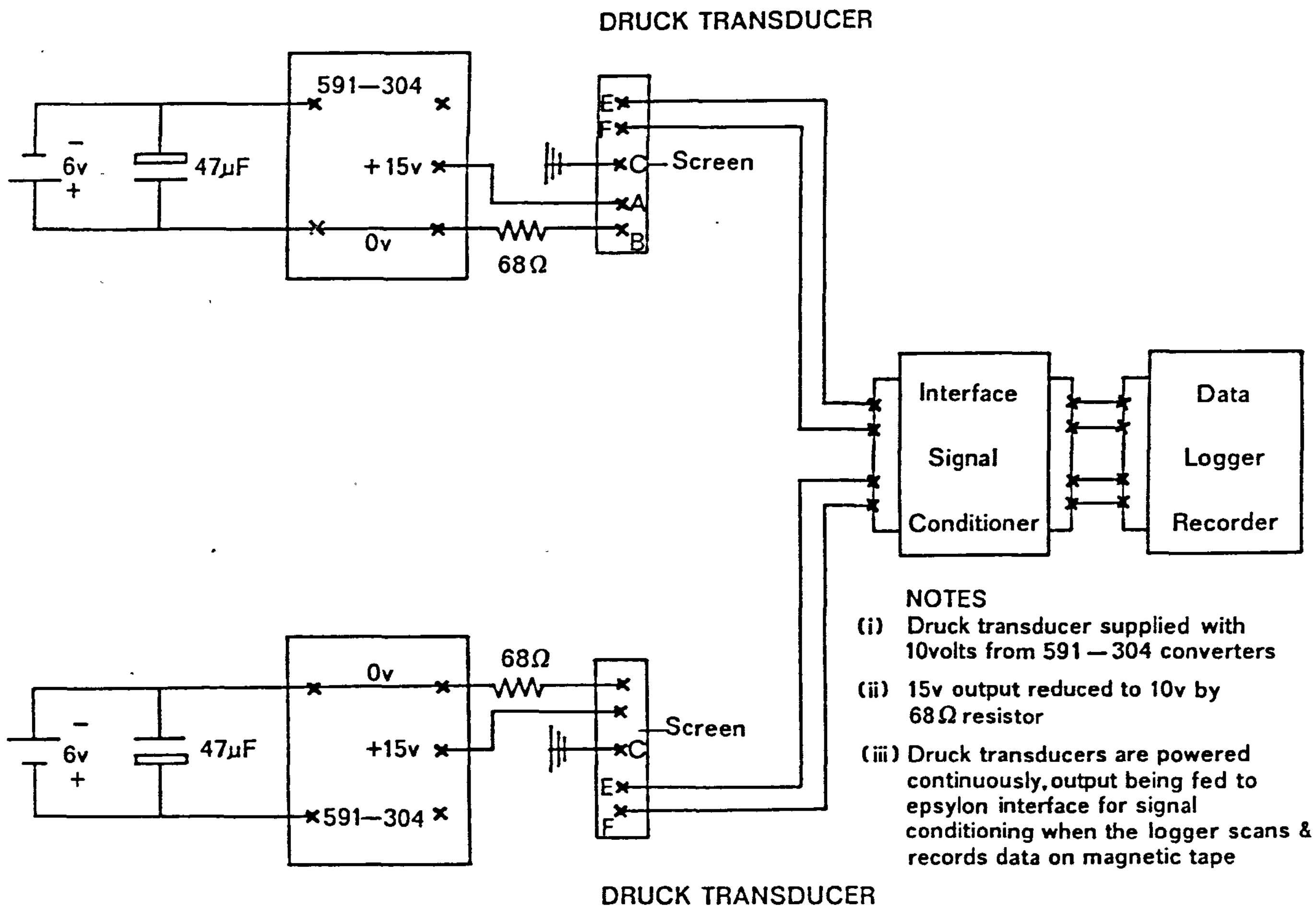



FIGURE 5.18

Interface designed to control the pressure transducers energising battery - transducer link piezometers in this study misconstrue porewater pressure values but also that such errors are present in other similar ground investigations. Standpipes use relatively large diameter inflexible pipes likely to promote and accentuate errors due to fissuring. Pressure transducers on the other hand are attached to the recording device via a thin flexible cable, giving less scope for such inaccuracy. It is suggested therefore,that readings from standpipe piezometers and similar equipment may well over estimate phreatic conditions although more detailed research is clearly required to confirm this.

### 5.4.3 Soil Moisture

The soil moisture content of a mudslide, particularly variations throughout the year, is important relative to the liquid and plastic limits. Other geotechnical properties and stability characteristics also depend on the soil moisture, as can rates and different types of movement (Hansen, 1984). Many standard measurement techniques are available (BS 1377, 1975; Curtis & Trudgill, 1974) but for this study it was decided to use the Wallingford Neutron Probe (Bell & McCulloch, 1969; Visvalingham & Tandy, 1972). This is a precise field method of measuring soil moisture. While many techniques involve material extraction and laboratory procedures, using a Neutron Probe does not. Consequently, there is no associated continuous disruption to the mudslide surface following access tube installation. The use of the Neutron Probe has not been reported in previous mass movement studies and it was felt that this study provided an ideal opportunity to utilise the technique. Other methods of soil moisture determination provide point data but this technique permits the calculation of a complete moisture profile through the mudslide. Also, the probe measures over a sphere of influence and consequently the results will minimise potential error due to stratigraphic variations.

Access tubes were constructed and installed following Institute of Hydrology guidelines (Bell, 1976). The bottom end of each tube was placed below the slip surface (Figure 5.19). Five tubes were installed at intervals up the mudslide, on the distance-angle survey transect. A standard count was obtained before and after each set of field measurements by suspending the Probe in an access tube installed in a drum of water. Control measurements were taken before and after each field visit, with readings noted down and up each tube at 10 cm intervals from 20 cm below the ground surface (Plate 5.2) using a 16 second count rate. The mean result was taken as the count rate. Other procedure followed Institute of Hydrology guidelines (Holdsworth, 1970; Institute of Hydrology, 1981) and data processing was conducted using a purpose-written micro-computer program.

Despite the advantages of this technique, previous research recognises a number of sources of error (Bell, 1976; Bell & Coles, 1967; Grant, 1975; Hewlett et al., 1964; Institute of Hydrology, 1981), some of which are pertinent to this study. The count rate reflects ground moisture content in the surrounding 15 cm of material. The technique is therefore limited in precisely locating changes, such as those across the slip surface for example. Also, changes in material density can affect the results and additional care is therefore needed with access tube installation.

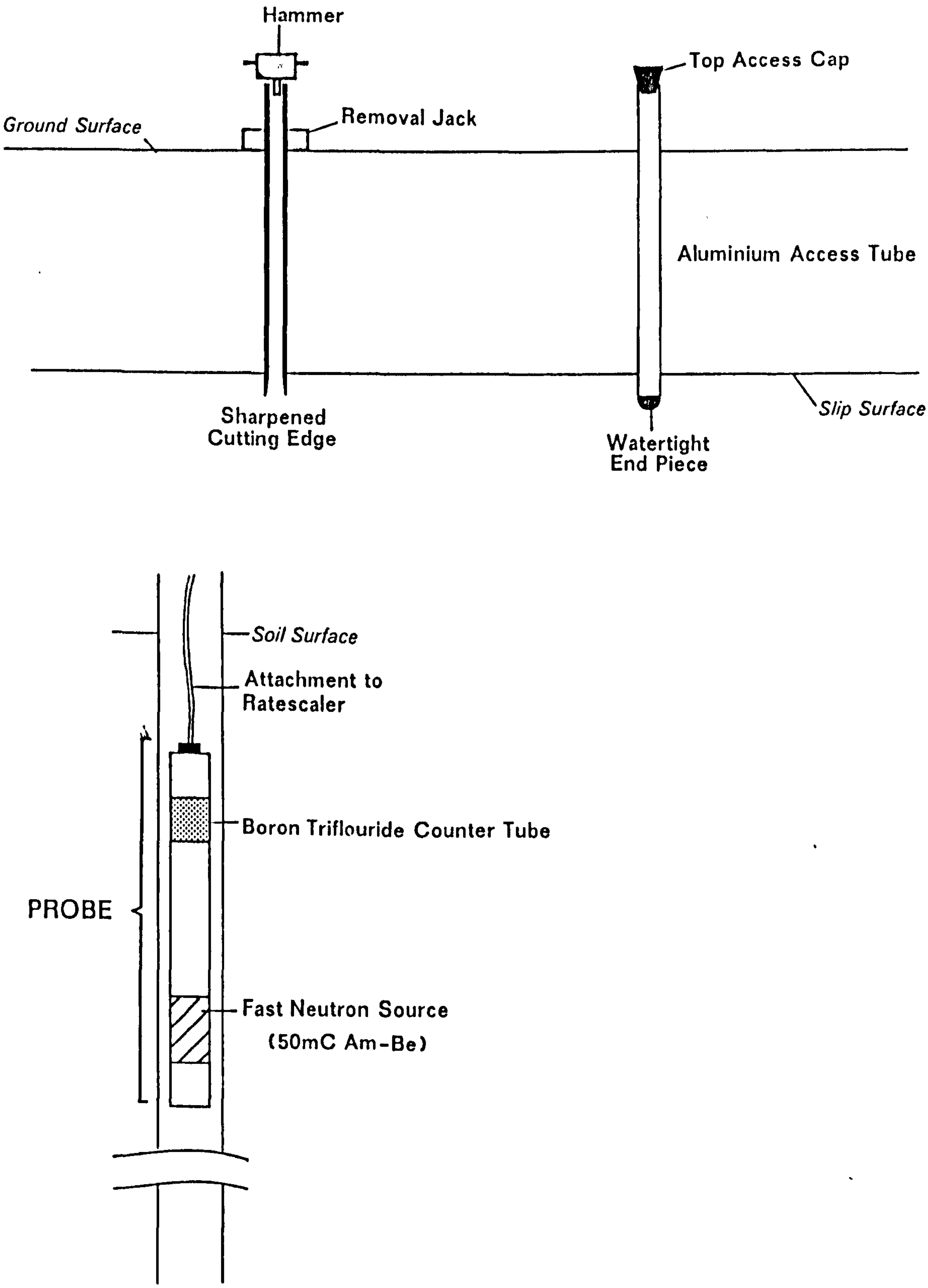



FIGURE 5.19

Details of Neutron Probe tube installation

PLATE 5.2

Groundwater measurement using Neutron Probe

### 5.4.4 Climate

Climate and in particular precipitation provide energy for the most important forces on hillslopes, those of static and moving water (Carson & Kirkby, 1972). There is much evidence to suggest that climate plays the dominant role in controlling natural slope stability (Brunsden, 1973; Campbell, 1974; Craig, 1977, 1985; Crozier, 1969; Hutchinson, 1970; Prior & Eve, 1975). Precipitation is particularly important, increasing soil moisture content, ground-water levels and porewater pressure. Initially precipitation was monitored using a 5 ml tipping bucket rain guage and digital event recorder (Plate 5.3). This was sited following standard procedure (Ward, 1975). Bucket tips were recorded on magnetic tapes which were returned to the laboratory for translation using a purpose written micro-computer program. Results were used to study the relationships between precipitation, soil moisture and movement. A detailed climatological record was obtained at Worbarrow Bay using an Automatic Weather Station (Lees _et al._, 1974) (Plate 5.4). Seven sensors measure wind direction, wind run, solar radiation, net radiation balance, rainfall, air temperature and depression (Table 5.8).

A Microdata M200L data logger with M200U interface unit recorded results. Data was logged on magnetic tape, translated and analysed using a PDP8 computer. The program converted raw data into listings of hourly and daily averages (Templeman, 1978). The weather station had to be positioned slightly inland from the coast to prevent vandalism. Although there is some climatic variation between the cliff edge and the installation site it was not sufficient to

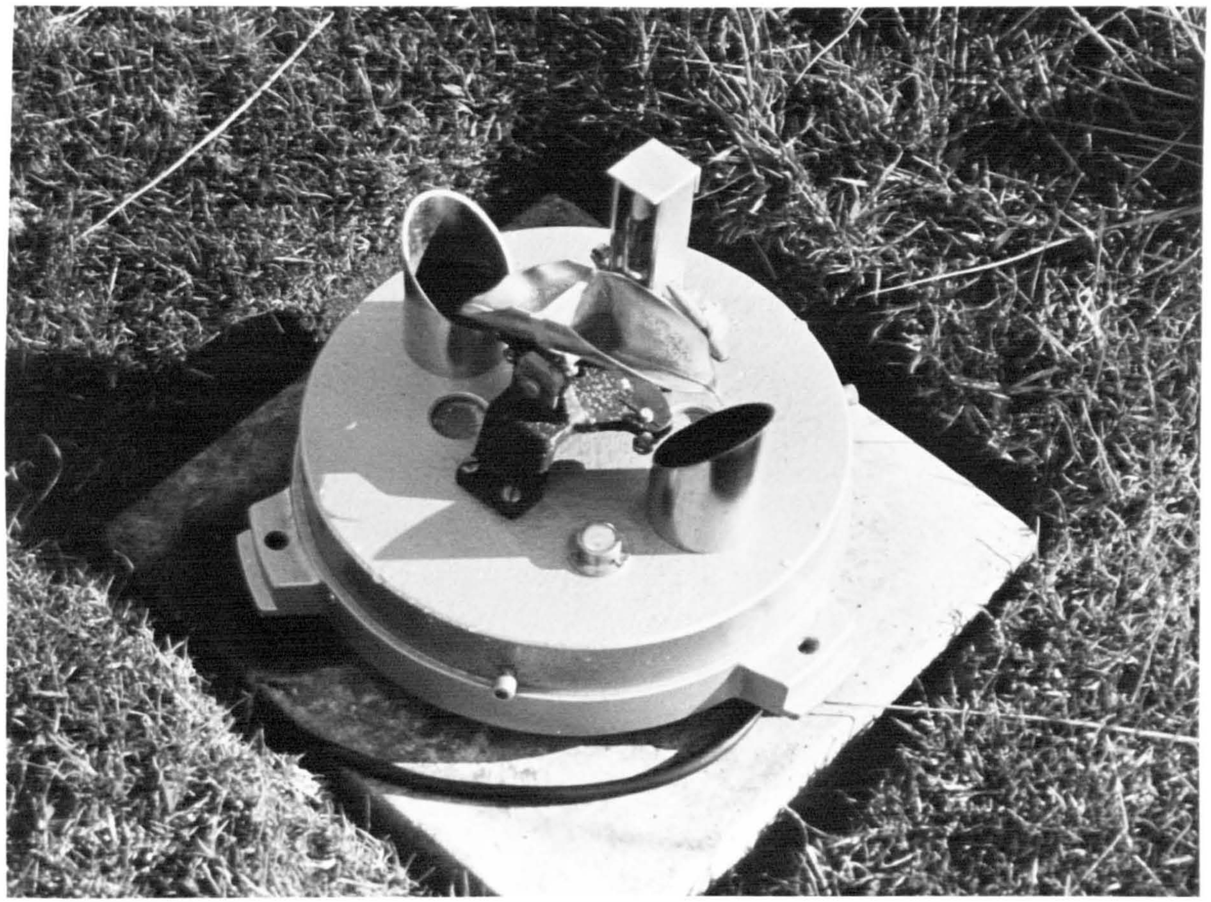
PLATE 5.3

Tipping bucket raingauge used to monitor precipitation

(a) raingauge complete with surround

(b) rain gauge with surround removed illustrating
tipping bucket mechanism

A



B

PLATE 5.4

Automatic Weather Station installed at Worbarrow Bay

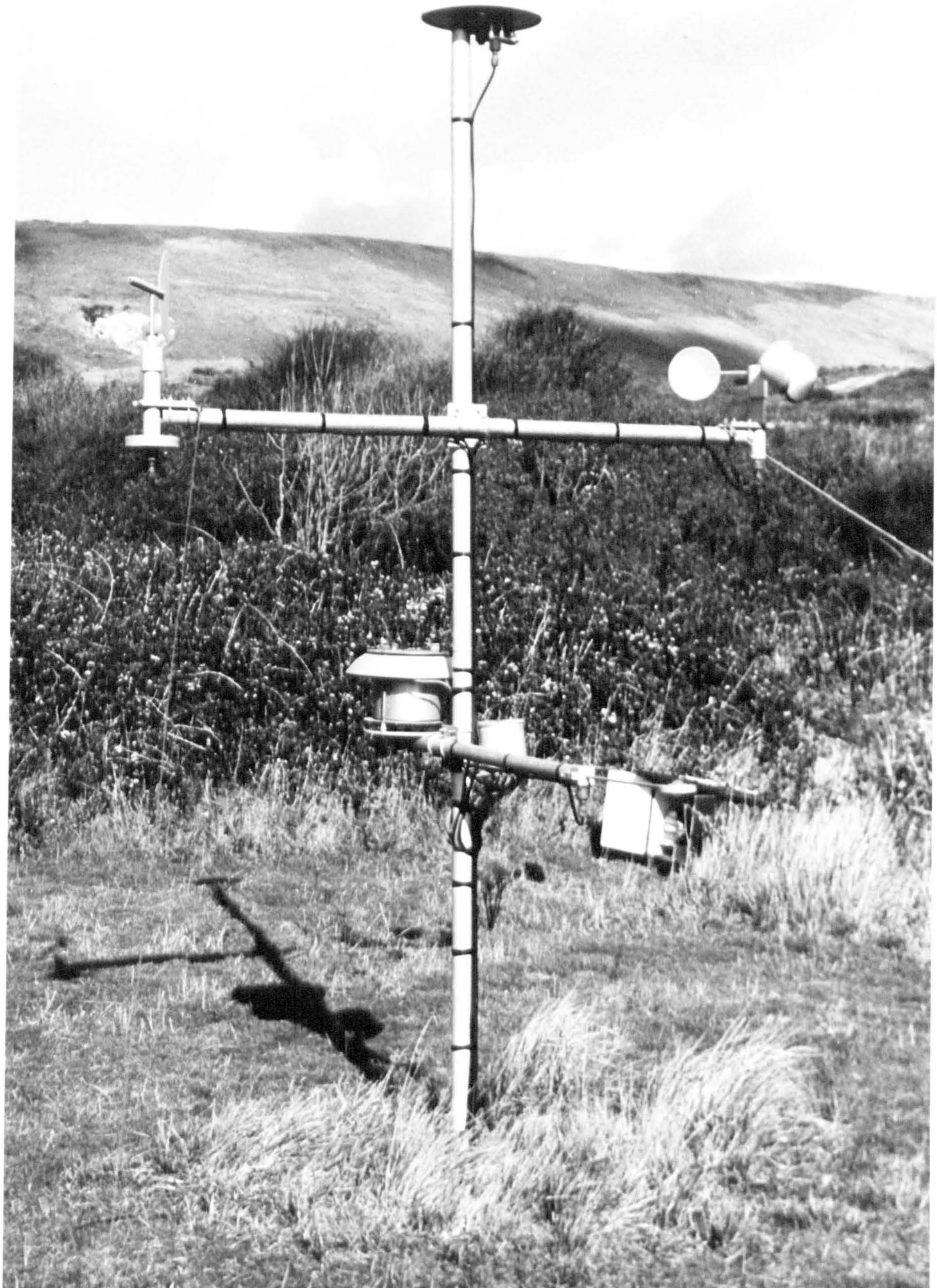


TABLE 5.8

Automatic Weather Station Technical Details

| Technical Details | Radiometer | Solarimeter | Rain Gauge | Anemometer | Wind Vane |
|---|---|---|---|---|---|
| Transmission Range | 0.3-80 m | 0.3-2.5 m | - | - | - |
| Output | 5mV/gcal/ $cm^2$/min | - | - | - | - |
| Resistance | 6Ω | 10Ω | - | - | 1156Ω |
| Sensitivity | - | 8mV/gcal/ $cm^2$/min | - | - | - |
| Accuracy | - | ± 1% | ± 1% | - | ± 10% |
| Linearity | - | 1% | - | - | - |
| Response time | - | 99%/30 sec | - | - | - |
| Capacity | - | - | 5 ml | - | - |
| Contact Rating | - | - | 0.5A 24V(DC) | - | - |
| Cup Diameter | - | - | - | 300 mm | - |
| Sensor | thermopile | thermopile | mercury wetted contact switch | reed switch | multiple reed switch |
| Range | - | - | - | 2-250 $kmh^{-1}$ | |

invalidate the results. This is seen by comparing precipitation recorded by the weather station and the tipping bucket rain gauge located on the edge of the cliff (Table 5.9). Even where totals differ,the change in precipitation is similar between sites, and the difference not great enough to exert a significant variation on the slope stability.

### 5.4.5 Surface Movement

Mudslide movements are generally related to the seasonal rainfall patterns and pore pressures (Brunsden, 1979). Hutchinson (1982) notes that 'movements may vary significantly and should be monitored throughout the period of investigation'. Previous attempts to monitor movement appear to have taken one of three forms (Table 5.10); using historical records, maps and air photographs (Arber, 1941, 1973; Conway, 1976 for example); installing surface markers and surveying (Balteanu, 1976; Brunsden, 1973; Crozier, 1970; Prior et al., 1968; Crozier, 1970) and using chart recorders (Prior & Stevens, 1971). Details of slope movements and cliff retreat,measured from maps and air photographs, have previously been presented (chapter III). At all sites, networks of fixed points were installed and surveyed around each mudslide. Lines of six inch nails and mudguard washers were positioned at 1 m intervals across the mudslide. The end nails in each row were located off the slide. Remeasurement involved surveying the fixed network to check for movement likely to result in errors and surveying each pin to calculate downslope movement.

TABLE 5.9

Rainfall differential between cliff edge and automatic weather station at Worbarrow Bay

| Tape No | D.E.R. Bucket Tips | A.W.S. MNI Rainfall |
|---|---|---|
| 1 | - | 4 |
| 2 | 14 | 39 |
| 3 | 19 | 13 |
| 4 | 56 | 30 |
| 5 | 63 | 29 |
| 6 | 43 | 53 |
| 7 | 3 | 10 |
| 8 | 34 | 19 |
| 9 | 48 | 25 |
| 10 | 13 | 15 |
| 11 | 38 | 37 |
| 12 | 9 | 6 |
| 13 | 64 | 46 |
| 14 | 27 | 16 |
| 15 | 49 | 24 |
| 16 | 24 | 13 |
| 17 | 0 | 0 |
| 18 | 14 | 9 |

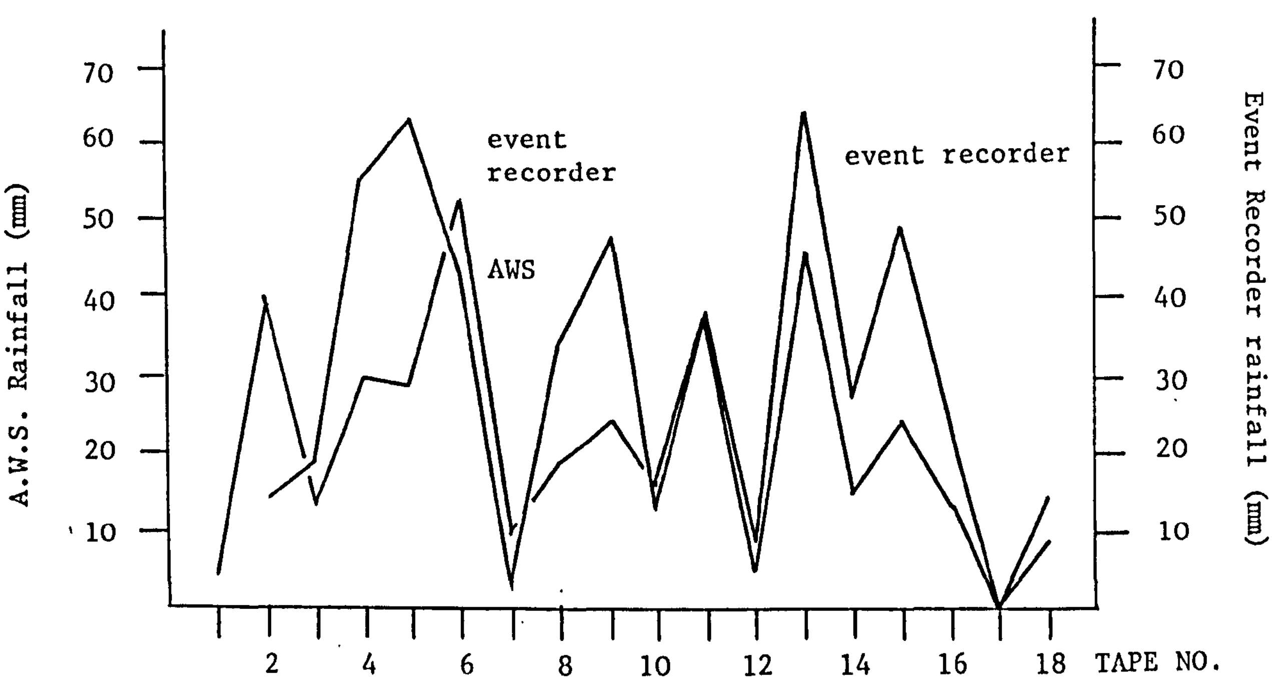


TABLE 5.10

Previous mass movement studies and techniques used to measure downslope material transport

| Report of Technique | Date | Location of Study | Technique Used |
|---|---|---|---|
| Craig | 1981 | Minnis North, E Co Antrim N Ireland | Painted line across mudslide |
| Hutchinson | 1970 | Beltinge, Kent, UK | Wood pegs |
| Van Moos | 1953 | Stoss, Switzerland | Stakes |
| Fukouka | 1953 | Mt Chausu | Stakes |
| Crandell & Varnes | 1960 | Slumgullion U.S.A. | Stakes |
| Auger & Mary | 1968 | Normandy, France | Stakes |
| Skempton & Hutchinson | 1969 | Beltinge, Kent, UK | Stakes |
| Prior *et al.* | 1971 | Minnis North, E Co Antrim N Ireland | Stakes |
| Hutchinson & Bhandari | 1971 | Isle of Wight, UK | Stakes |
| Brunsden | 1973 | Stonebarrow, Dorset, UK | Wood plates and pins |
| Hutchinson | 1970 | Beltinge, Kent, UK | Time lapse photography |
| Tan | 1983 | Minnis North, N. Ireland | In-house automatic logger |
| Prior *et al.* | 1971 | Minnis North, N. Ireland | Modified Monroe recorder |
| Craig | 1981 | Minnis North, N. Ireland | Monro chart recorder |

A number of problems were encountered. At Stair Hole two of the fixed points were removed. Accurate remeasurement of these markers was essential. At some sites pins were removed, while in other cases movement obscured the head of the nail from the theodolite, preventing direct measurement. Measurements in these cases were made to a survey pole placed on the head of the nail and maintained in a vertical plane using an attached spirit level. In particularly wet winter months, planks had to be used to obtain access to the pins at Mupe and Worbarrow, to minimise direct contact with the ground surface and consequent material and pin disruption. When mudslide surges occurred some of the pins were lost, and establishing patterns of movement became limited.

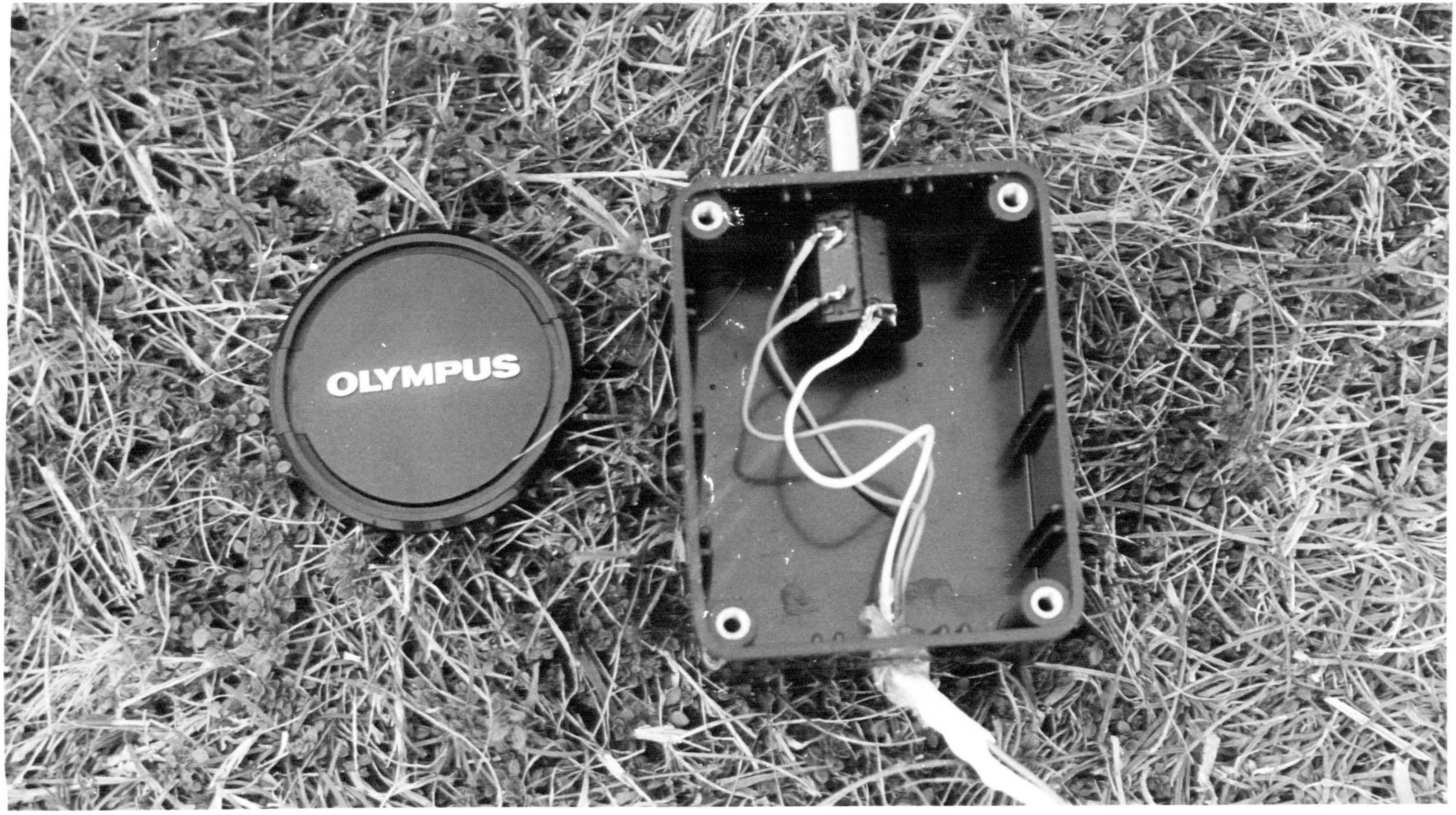
Detailed monitoring was conducted at Worbarrow Bay. Brunsden (1984) notes that 'it is rarely possible to derive inter-event measurements and only cumulative or average investigations of activity since an event can be made'. He goes on to say that attempts to obtain a continuous record have been made. Prior and Stevens (1971), for example, developed mechanical recorders in the form of modified Monroe water level recorders. However, Brunsden concludes that there is much scope for improvement of these techniques, in particular by using electrical recorders and microprocessors. No results of such systems have yet been published although various types of apparatus have been proposed (Armstrong et al., 1985). As part of this research a microprocessor based continuously recording electrical system for monitoring slope movements was designed and installed. Developments included movement sensors, the sensor-data logger link, and sensor mounting. Modifications to the data logger were also required.

Movement was monitored using rotating potentiometers. Pins 6" long were installed across the mudslide surface and connected to the potentiometers using nylon wire. The potentiometers were secured within a protective A.B.S. box with the potentiometer shaft protruding through one end of the surround (Plate 5.5). Eight of these sensors were attached to a gantry suspended over the mudslide. A pulley wheel was inserted over each potentiometer shaft to carry the nylon thread. These were machined to precise dimensions (Figure 5.20) to give a direct linear relationship between the change in resistance across the potentiometer, the data logger record and downslope movement (Figure 5.21). The pins were located approximately 2 m downslope from the gantry (Plate 5.6). A twelve channel data logger was used to record changes in resistance across the potentiometer (Plate 5.7) with interface cards in each of the twelve slots to perform the following functions: channel 1, a scan marker required by the software to identify the start of a recording period; channels 2 to 9, cards designed specifically for this study and used to interface the potentiometers with the data logger (Figure 5.22); channels 10 to 12, real time cards for recording the time of each scan. Data was recorded on magnetic tape in binary format and information transferred from tape to disc using a purpose written micro-computer program. Translation was checked by printout provided during data transfer.

Problems were initially encountered in keeping the nylon wire on the pulley. This was overcome by pumping axle grease into the pulley groove and fixing perspex wipers with an attached foam pad to the end of the wiper resting against the pulley groove. Difficulties were encountered due to flexure of the gantry under

PLATE 5.5

Design of surface movement sensors


OLYMPUS

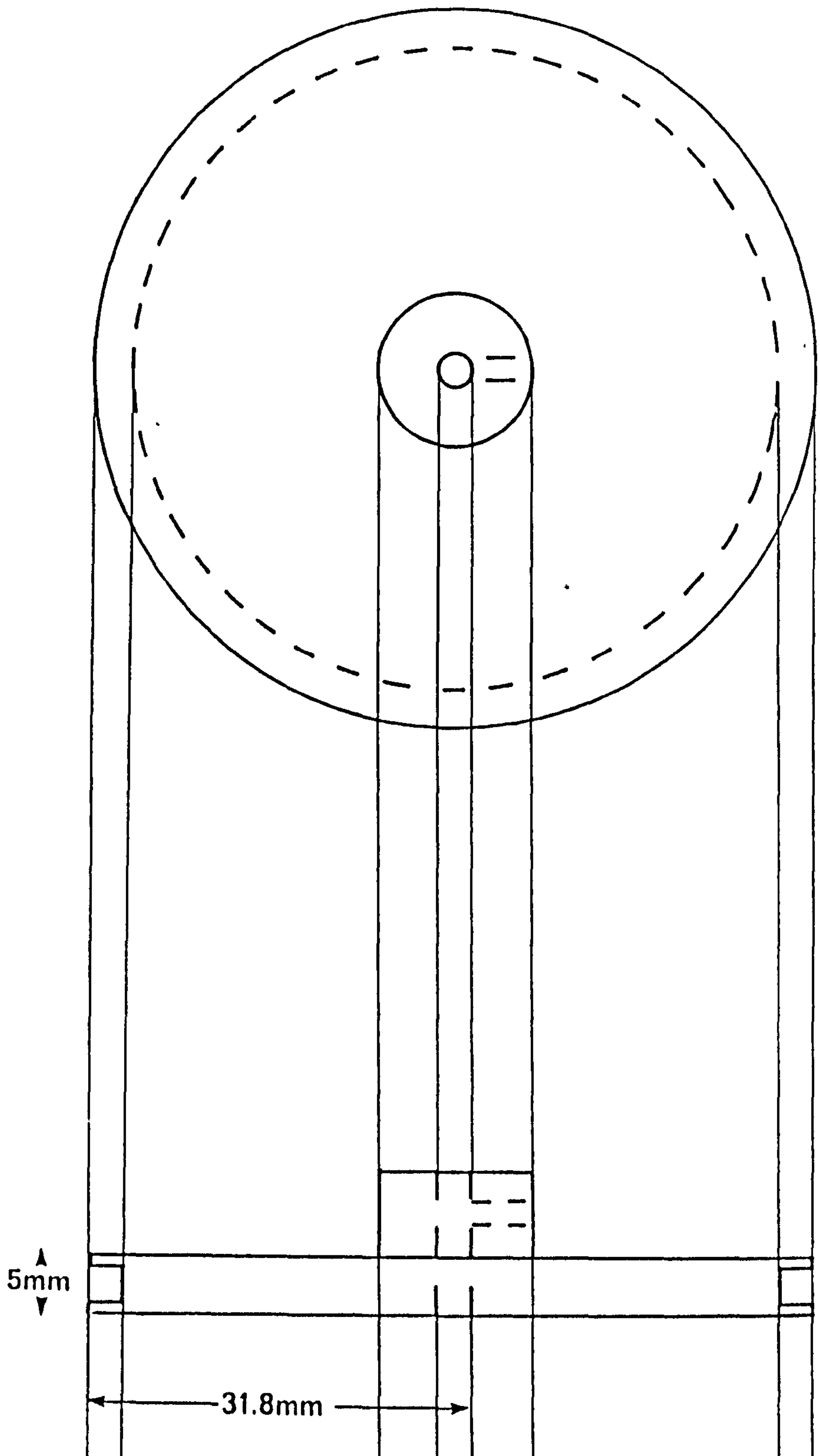



FIGURE 5.20

Pulley wheel design for surface movement sensors

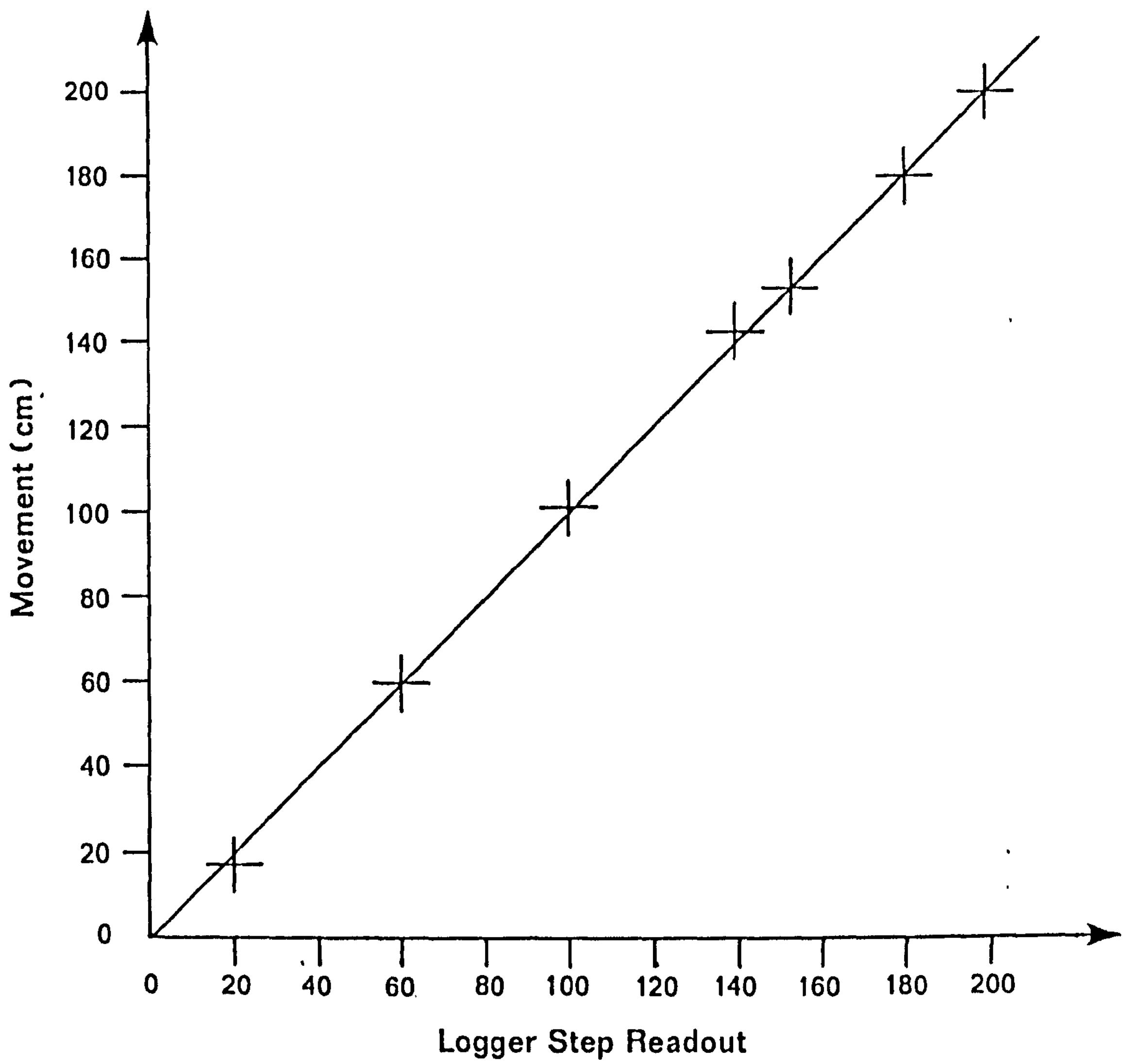



FIGURE 5.21

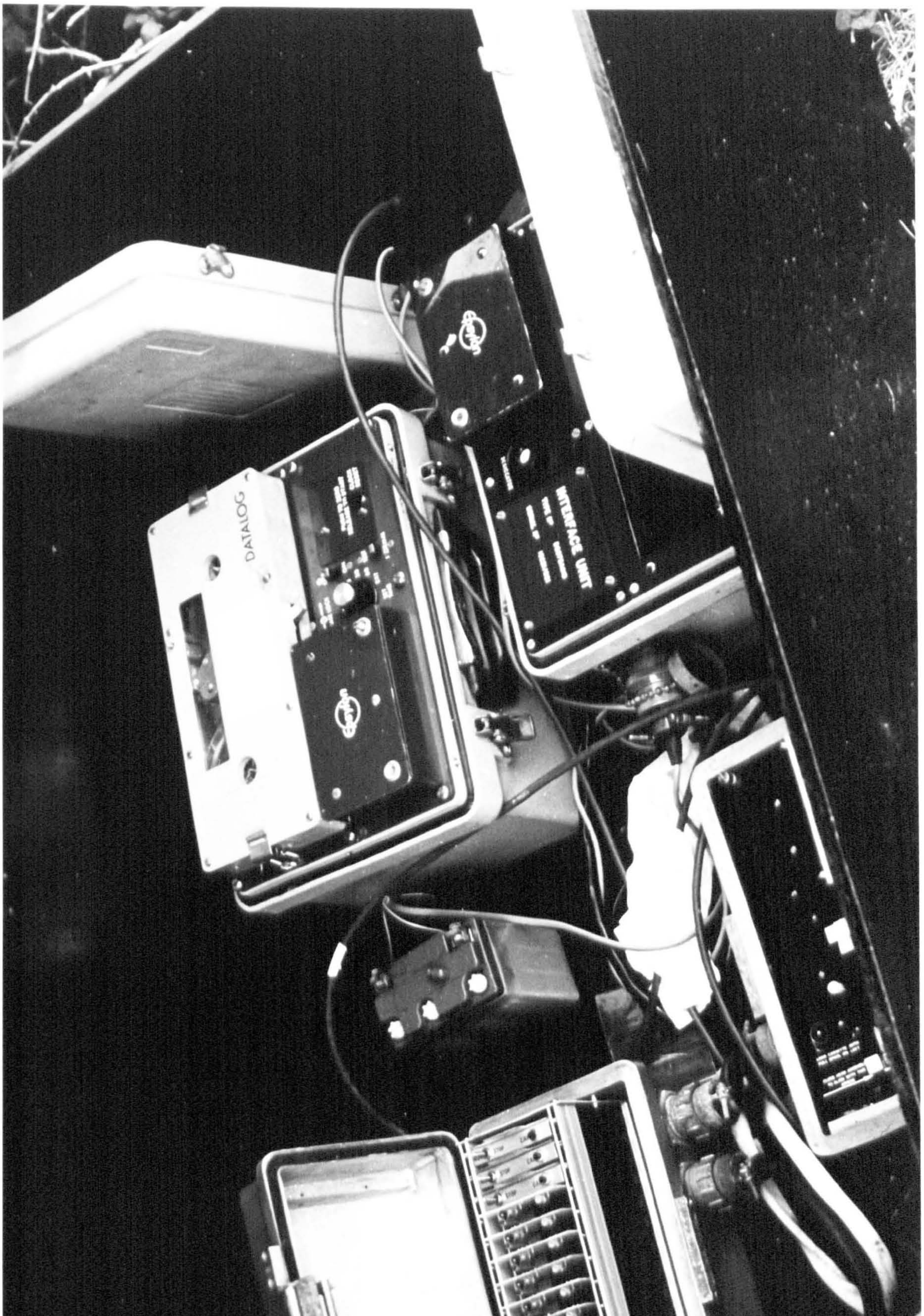
Calibration of surface movement sensor
potentiometers against downslope movement

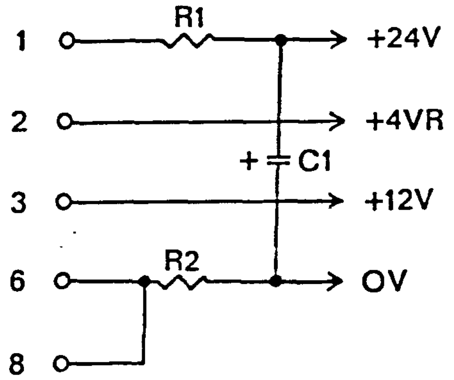
PLATE 5.6

Surface movement sensor attached to supporting frame

PLATE 5.7

Data logger used for recording slope movements

DATALOG
INTERFACE UNIT




| TRIMPOT | FUNCTION |
|---|---|
| VR1 | SENSOR CALIBRATION |
| VR2 | OFFSET TRIM |
| VR3 | GAIN TRIM |

When Impendance of Sensor is less than 1k$\Omega$
Buffer Amplifier is not required.

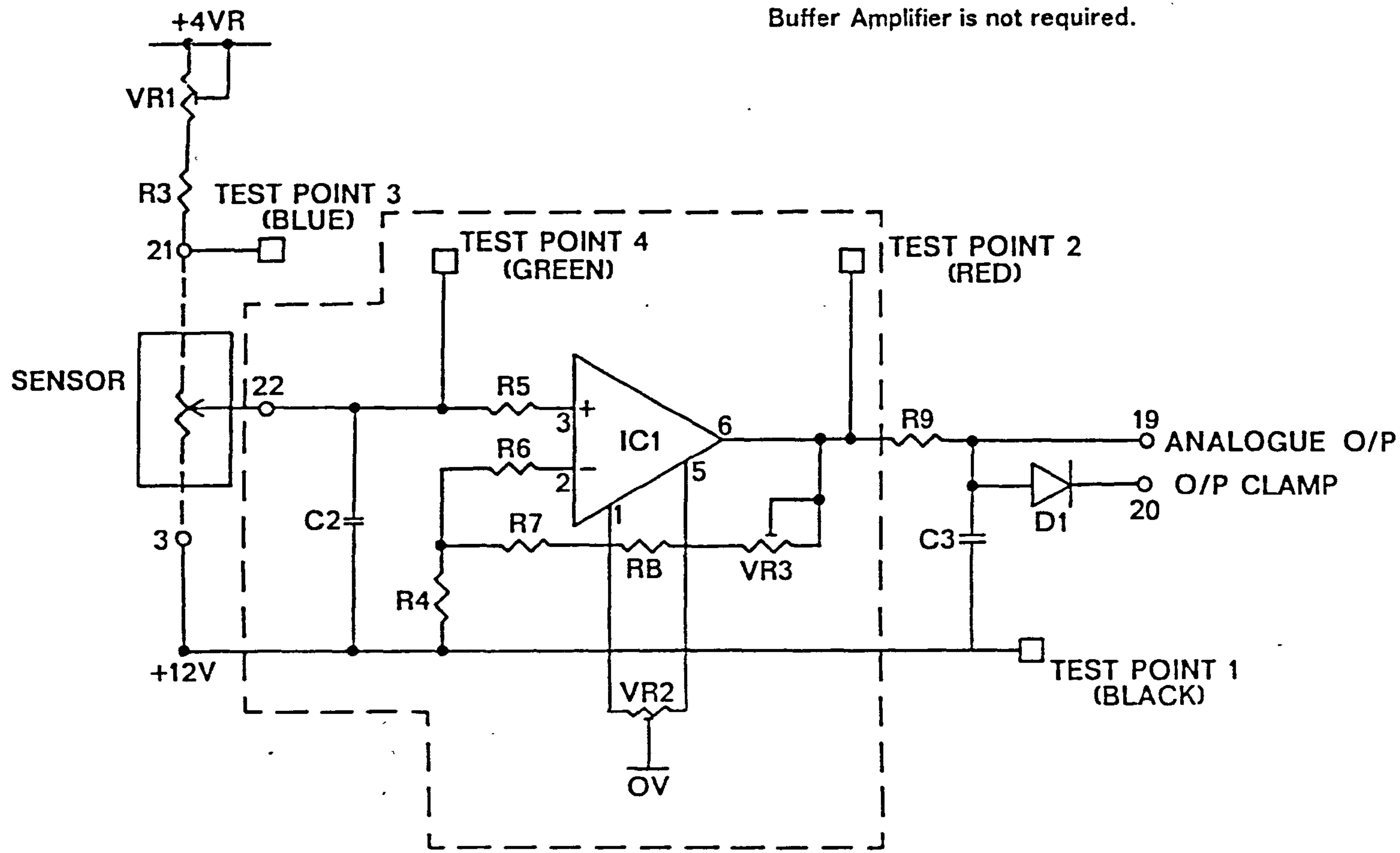



FIGURE 5.22

Data logger inteface card
designed for the monitoring
of surface movement

strong winds. Cross-braces on the metal frame and guy ropes were used effectively to minimise this. The results were also checked against the automatic weather station wind run data for changes in the record, thought to be due to gantry vibration rather than mudslide surges. Finally, due to the mobilisation of a previously stable part of the slope, the first frame suspending the potentiometers and A.B.S. boxes was destroyed during a major mudslide surge during the winter of 1984/1985. Equipment had to be reinstalled and rewired.

### 5.4.6 Subsurface Movements

Of the techniques available for measuring subsurface displacements (Table 5.11) the alternatives available to this study were restricted. Consequently, despite developments in this field (Brunsden, 1973, 1984; Hutchinson, 1970; Shannon et al., 1962) problems were experienced in this part of the research design. The installation and recovery of apparatus would have been impossible without extensive excavation likely to cause errors to other results. Measurements were attempted at all sites using the film-track potentiometer and galvanometer assembly described by Brunsden (1973). Three 5 cm diameter flexible spiral wire plastic bound tubes were installed vertically on each mudslide. Lowering the monitoring device down the tubes, however, caused major distortions to the vertical profile making it impossible to assess the extent to which results were due to either true mudslide movement or the capability of the galvanometer assembly to distort the flexible tube while taking readings. Consequently, these results were regarded as unsatisfactory.

TABLE 5.11

Techniques available for monitoring mudslide sub-surface displacements

| Report of Technique | Date | Location of Study | Technique Used |
|---|---|---|---|
| Skempton & Hutchinson | 1969 | Beltinge, Kent | Flexible tube - loaded leaf inclinometer |
| Prior *et al*. | 1971 | Minnis North, N Ireland | Re-excavated flexible tubing |
| Craig | 1981 | E Co Antrim, N Ireland | Re-excavated flexible tubing |
| Hutchinson | 1970 | Beltinge, Kent | Helical flexible tube and strain gauge inclinometer |
| Young | 1960 | | Young's pit |
| Selby | 1968 | | Buried cones and posts |
| Brunsden | 1973 | Stonebarrow, Dorset | Helical flexi-tube and film track potentiometer |

## 5.5 TEMPORAL SAMPLING FRAMEWORK

Brunsden & Thornes (1978) note that little attention has been given to the problem of experimental design for temporal sampling frameworks. This is particularly reflected in mass movement studies, which are hampered by an inability to measure long term climatic cycles and difficulty in establishing to what extent measured movement reflects antecedent conditions, due to the time lag between critical changes in the controlling variables and form modification. Investigations are usually conducted for one year to establish short term climatic influences. However, there is no guarantee that a single set of annual measurements will be representative of mean conditions. Also,cumulative results often mask infrequent individual events which can be of great significance. This is particularly relevant in mass movement studies where single events can account for 80-90% of the total movement. This suggests that a low sampling density is required for periods when a slope is likely to undergo steady change due to regular events,while for the low frequency/high magnitude events, a high sampling density is required.

The sampling framework for this study had to be designed around a number of constraints. Limited financial support was available for field visits. During the first 12 months of this study, land access to Mupe and Worbarrow Bays was restricted to occasional week-ends, public holidays and extended periods in December, April and August. The field monitoring programme forms part of a larger study and time had to be suitably apportioned to different parts of the laboratory and field schedule.

Using the described standard techniques, controlled observations were taken at monthly intervals. These results provide details of broad changes throughout the season. Occurrences such as the high magnitude rare event were identifiable but not at a precise point in time. This data was used to identify the time of important mass movement events, thus permitting a greater sampling frequency during the critical weeks of the following year. Thornes and Brunsden (1978) note that 'continuous observation is the only real way to catch the rare but sometimes important event'. Consequently as many variables as possible were continuously monitored at Worbarrow Bay using data loggers. Battery life and tape capacity between logger maintenance periods (Table 5.12) permitted the collection of results at 5 minute intervals, providing over 700 000 pieces of data. No evidence currently exists of other mass movement studies with this level of detail or resolution.

## 5.6 RESULTS FROM MONITORING AT ALL SITES

The results of the field investigations are best presented in two principal sections. Firstly, the data collected at all sites on a monthly time base will be considered, detailing specific characteristics relevant to this study. This includes particular aspects of variables such as soil moisture and relationships between different results including porewater pressure and slope movement. Seasonal and annual trends will be identified. Secondly, the data collected at Worbarrow Bay using the short sample interval monitoring programme will be considered. This data set is so large that it is impossible to present the results in their entirety. During much of the year these records provide no more information than is

TABLE 5.12

Estimated Data Logger battery life and tape length

I BATTERY

assuming battery capacity of 1 Ah operating time (OT):

$$OT = \frac{600t}{(n+1)1.3}$$

where $n$ = no. channels in use = 12
$t$ = time interval between scans = 10

$$OT = \frac{600 \times 10}{(12+1)1.3} = 14.79 \text{ days}$$

II TAPE

total capacity for C60 = 57,000 words

time to fill cassette TC:

$$TC = \frac{ct}{60(n+1)}$$

where TC = recording time in hours
$t$ = recording interval (minutes) = 10
$c$ = tape capacity = 57,000
$n$ = number of channels in use = 12

$$TC = \frac{57{,}000 \times 10}{60 \times (12+1)} = 30.4 \text{ days}$$

available from the monthly sampling framework. These results can therefore be ignored. Finally,data sets can be compared to assess the increased level of interpretation accompanying the higher sampling frequency and hence the detail lost in previous studies of a similar nature.

### 5.6.1 Climate

Of the data collected using the on-site Automatic Weather Station located at Worbarrow Bay and the tipping bucket rain gauge, wind direction, precipitation and the net hydrological flux are important. The latter encompasses all relevant parameters including temperature, evaporation, wind speed and relative humidity. Wind direction is important due to the coastal location and exposure of the cliffs to onshore winds. The record at Worbarrow Bay (Figure 5.23) showed strong bi-directional winds from the north-north-east and south-south-west indicating in the former case, a funnelling effect along the axis of the Tyneham Valley and in the latter instance strong onshore breezes. The winds blowing from land to sea are likely to be a function of local topography and will therefore not be as signficant at other sites. All cliffs are directly exposed to the dominant wind and accompanying characteristics such as sea spray.

Precipitation is best considered by comparing the data collected during this study with mean totals calculated from long term Meteorological Office statistics collected at Wareham. Mean monthly maximum precipitation occurs during November, December and January with minima in April, May and June (Figure 5.24). The

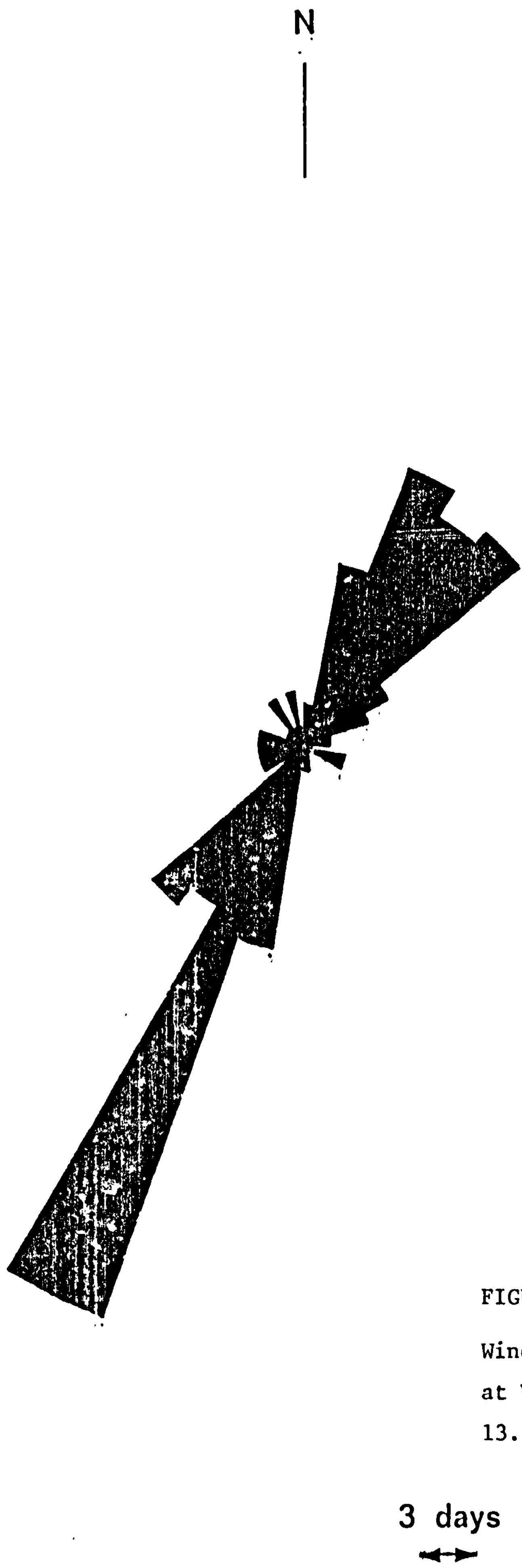



FIGURE 5.23

Wind directions recorded at Worbarrow Bay from 13.10.84 to 23.03.85

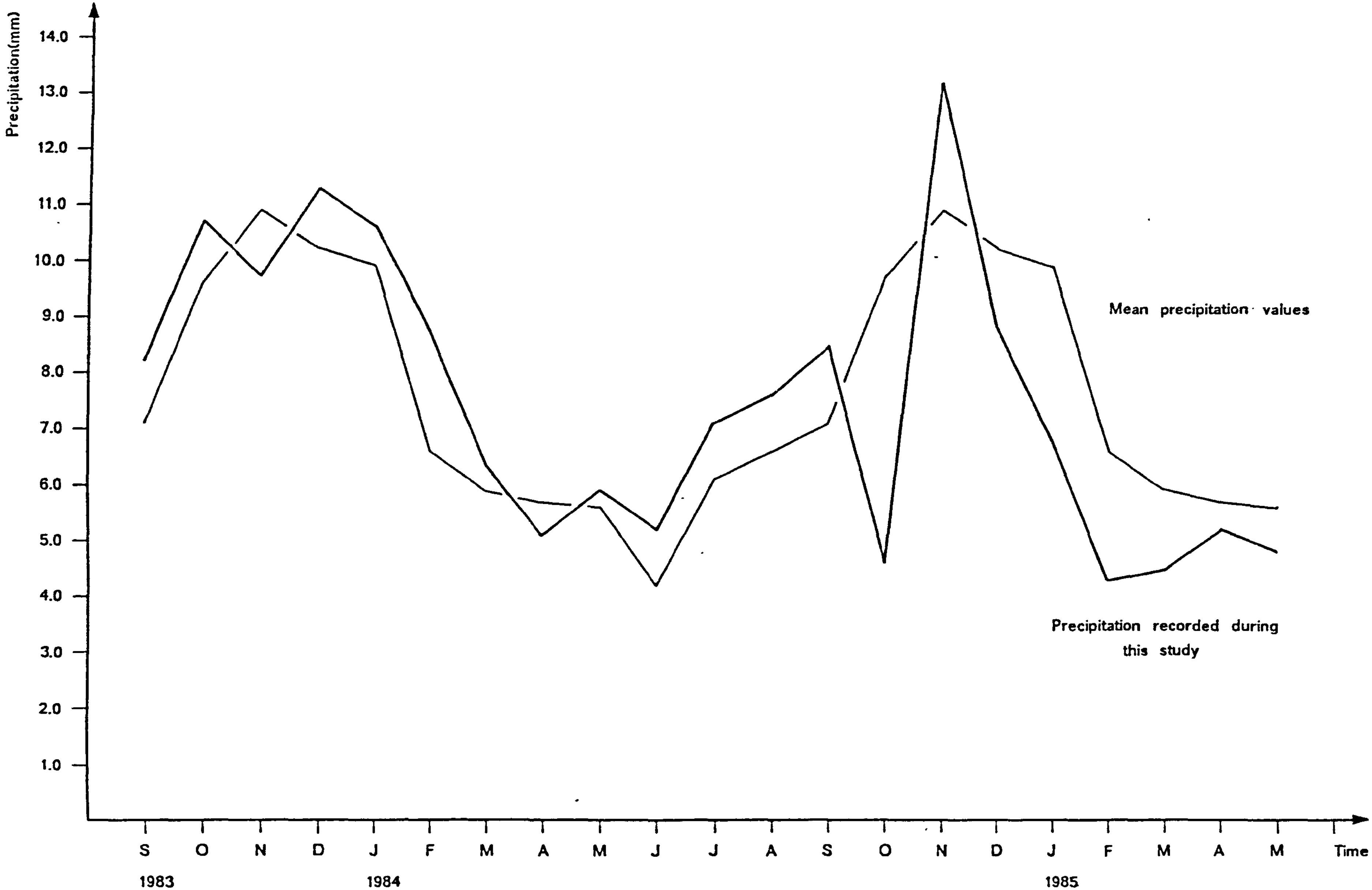



FIGURE 5.24 Total precipitation recorded during this study

collected data varies from this trend in a number of respects. The winter months of 1983/1984 received above-average precipitation for the time of year. Rainfall was in the order of 10 mm above mean totals for September, October and between December and March. Precipitation remained above-average for most months throughout the summer of 1984 until October. The reverse occurred during the winter of 1984/1985 with monthly rainfall totals well below mean Meteorological Office Statistics. Only November was wetter than usual, receiving an additional 22 mm above the mean monthly total. A trend therefore emerges of conditions which were wetter than usual at the start of the field monitoring program being followed by a drier than average winter in 1984/1985.

The monthly precipitation variations were reflected in the site hydrological flux calculated using the Penmann equation for estimating daily evaporation based on the heat budget and aerodynamic conditions (Figure 5.25). A water deficit at the beginning of the record changed sharply to a surplus between 13th - 20th October 1984, remaining positive until mid-February. The budget varied from peaks exceeding + 50 mm to values of almost zero. Maximum values in November reflected the abnormally high precipitation totals during that month, while on other occasions the hydrological flux remained high, although precipitation was below mean monthly figures.

A significant drop occurred in the water balance during the first and second weeks in February. This was thought likely to have an effect on slope movements which rely on high soil moisture contents and porewater pressures. This highlights the importance of

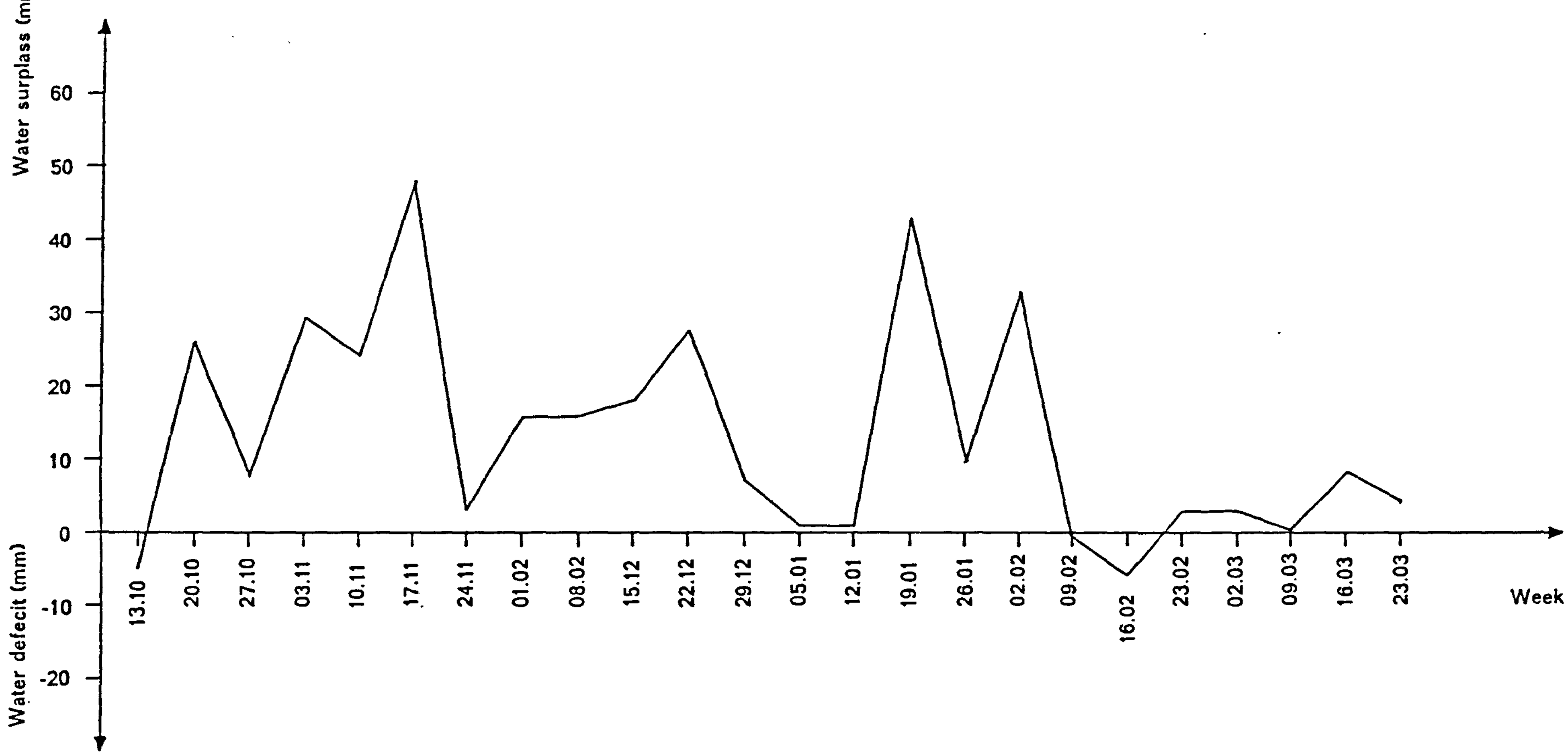



FIGURE 5.25

Water flux recorded at Worbarrow Bay from 13.10.84 to 23.03.85

considering more than just precipitation when examining interrelationships between climate and slope movements. Clearly a more comprehensive consideration of climatic variables will enhance an understanding of any causal relationship.

### 5.6.2 Rate of Input and Backslope Loading

Difficulties were encountered in measuring material detachment from the rear scar. Erosion pins would have yielded suitable results,particularly at Stair Hole where material is removed dominantly by surface wash,but as previously discussed (section 5.4.1) implementation of this technique at Stair Hole and Mupe Bay was not possible. The dowel pegs installed at the rear of each slope gave poor results. They were installed at one point behind each slide,making it impossible to measure detachment along the whole of the rear scar. The installation of a large number of pegs would have made them obtrusive, however, and human disruption would have undoubtedly have accompanied this. These installations also assume that detachment takes place along the rear scar,but this is not always the case. Although plans were made to keep quantitative records,no changes were recorded during the period. This highlights the inadequacy of the technique. Each rear scar was reviewed regularly and as no significant change was recognised at Mupe Bay and Stair Hole the effects of rear scar loading on movement were assumed to be insignificant during the survey period. Supply must be necessary, however, for the long term maintenance of the slide volume.

Investigations at Worbarrow Bay using erosion pins were successful. The results are presented as a grid, each square representing the 0.5 $m^2$ area around each erosion pin installed across the rear scar. Remeasurements were made at weekly intervals. Detachment follows a temporal pattern permitting the summarisation of results in three diagrams (Figure 5.26). Little change was recorded until 18th November 1984. Material dislodged before this date never exceeded 10 cm at any one point. Removal was by the weathering and detachment of individual particles and small flakes, rather than large blocks. Major change occurred between 18th -28th November 1984. Within two weeks much of the rear scar had retreated by 1.5 m, with totals exceeding 2.0 m in places. All material fell onto the rear of the mudslide. Consequently, much of the loading at the head of the slope occurred within a short time interval. Detachment probably occurred in a 'domino' manner, since the in-situ blocks undoubtedly relied heavily on adjacent units for support. Failure at one point was therefore probably followed by a knock-on effect. Most of the change occurred when precipitation totals were above mean values and, more significantly, during that period of the year which experienced the highest positive hydrological flux.

Following this event, little change was recorded during the remainder of the sampling period. Further detachment appeared to be in one of two forms; removal of individual particles and small flakes by subaerial weathering and secondly, the dislodgement of material left in a critical state following the previous movements. Some of the original installations registered little or no change. Those erosion pins at the extremities of the rear scar were in this category and are excluded from the figures to permit better representation of those

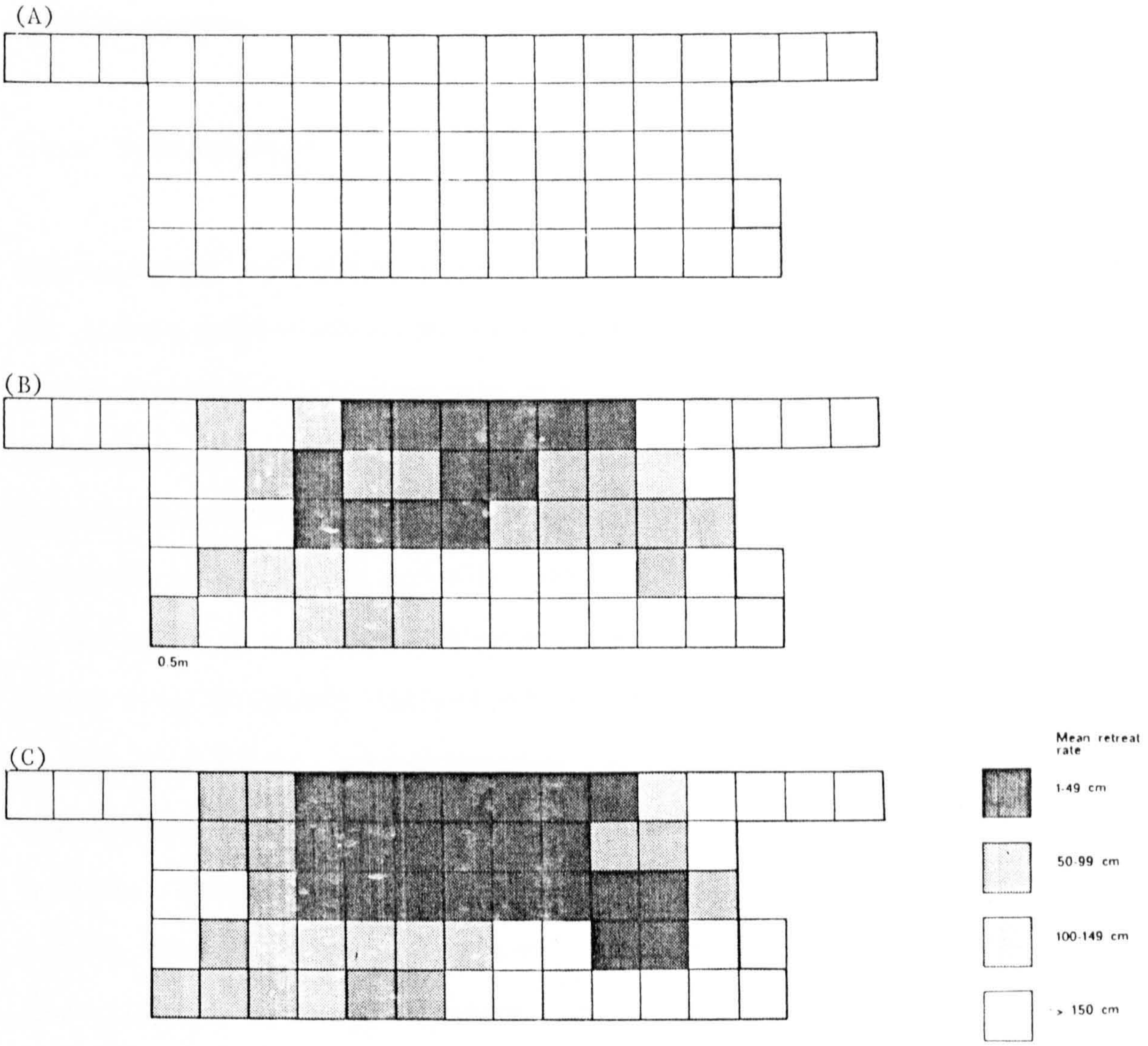



FIGURE 5.26

Spatial distribution of detached blocks from the rear slope scar at Worbarrow Bay

(A) From start of record to 17.11.84

(B) From 18.11.84 to 27.11.84

(C) From 28.11.84 to end of record

points where change did occur. All detached blocks fell onto the rear of the main mudslide, many maintaining their original morphology. Evidence of similar events suggest movement downslope is accompanied by disaggregation and eventual incorporation of the blocks within the mudslide matrix.

### 5.6.3 Soil Moisture

Results of the soil moisture survey conducted at Worbarrow Bay using the neutron moderation technique are presented as plots of moisture volume fraction (m.v.f.) against depth (Figures 5.27 - 5.31) for each access tube (Institute of Hydrology, 1981). M.V.F. variations through the profiles, temporal changes and spatial differences between different points on the mudslide require consideration. At the rear of the mudslide the m.v.f. increased rapidly from 20 cm - 30 cm depth, with relatively uniform values throughout the remaining profile in October. During November, the rear slope moisture profile changed, with distinctly higher m.v.f. values towards the ground surface. The moisture content of the material below 30 cm increased towards the end of the year however. The speed of such change no doubt relates to the infiltration rate of the material. The overall rise in the m.v.f. corresponds to higher precipitation totals and an increased positive hydrological flux. On 21st December 1984 access tube distortion due to slope movements prevented the measurement of a complete moisture profile. However, incomplete records suggested a drop in the m.v.f. below 30 cm following slip, probably due to the opening of fissures in the matrix permitting increased percolation, greater evaporation over larger surface areas and a reduction in the neutron count rate where winds are

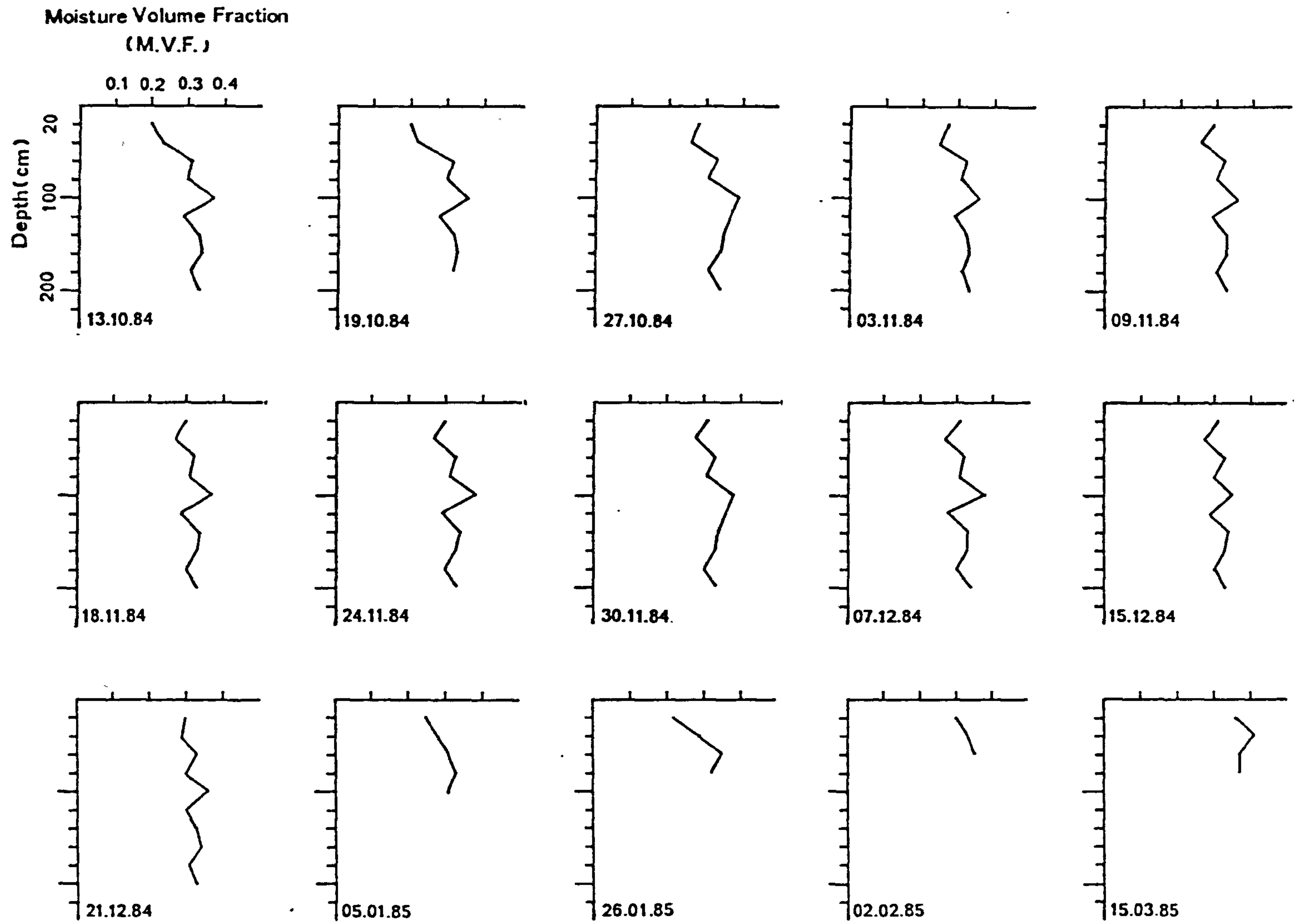



FIGURE 5.27

Neutron Probe results: top slope access tube

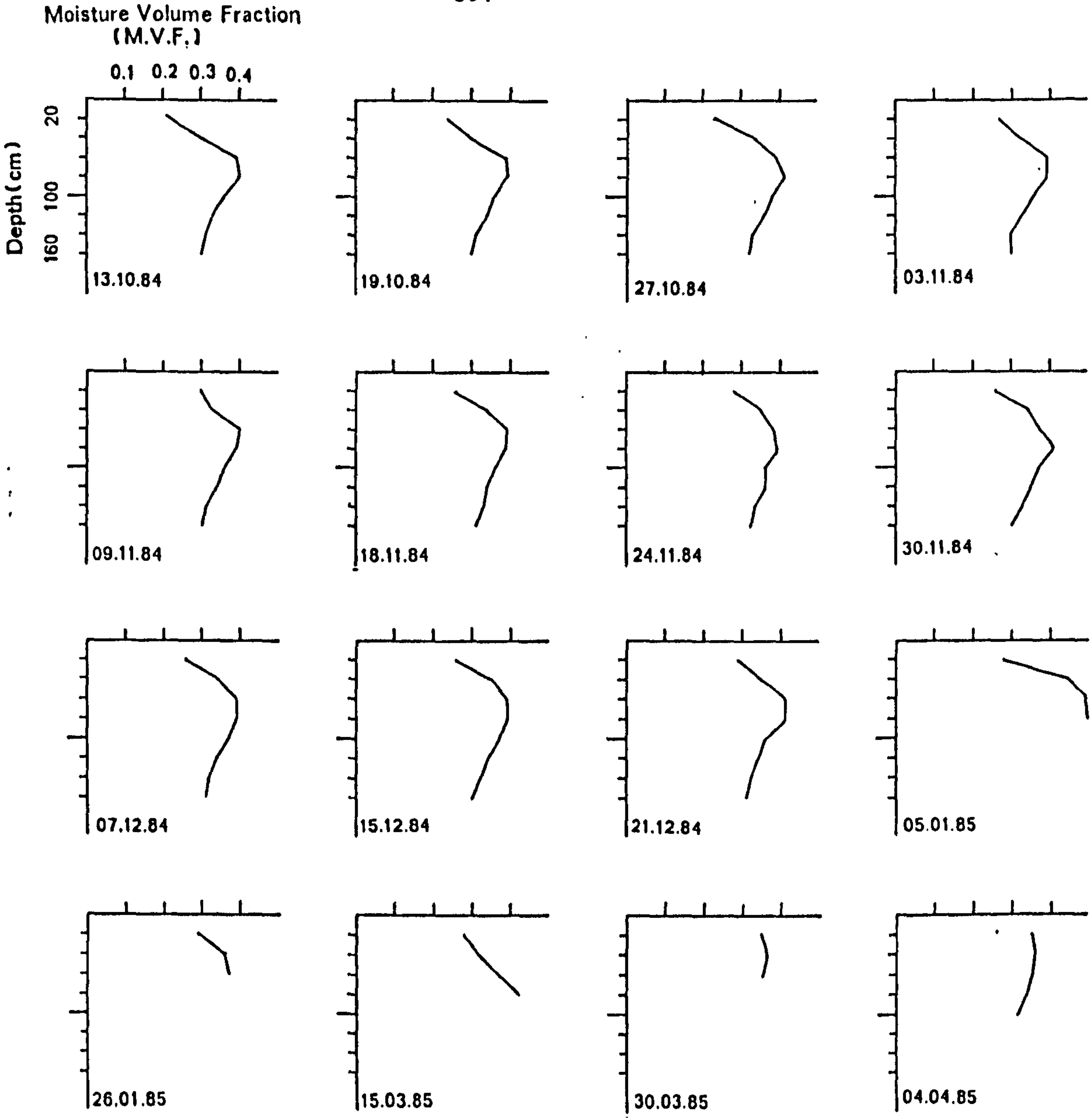



FIGURE 5.28

Neutron Probe results: upslope access tube

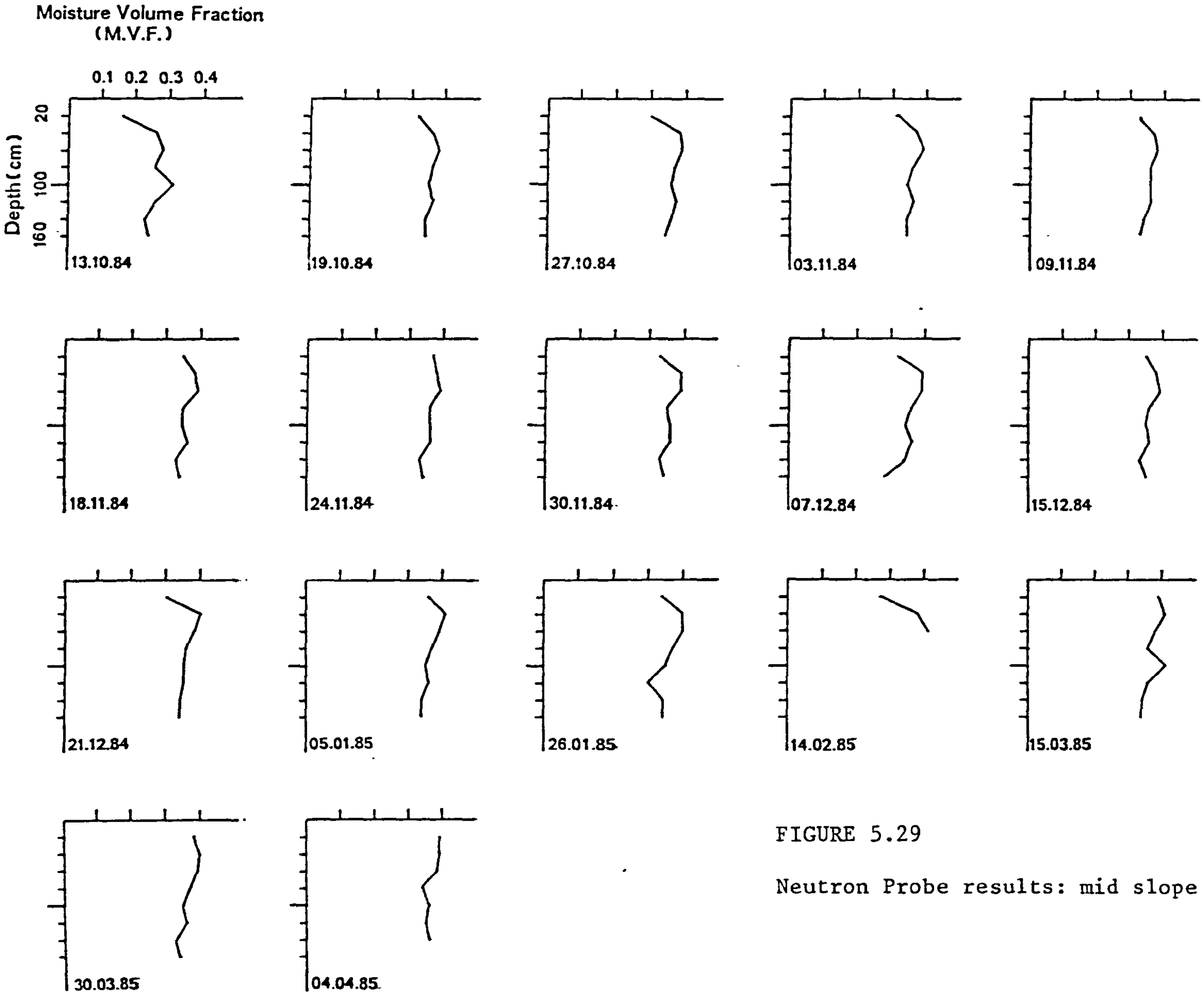



FIGURE 5.29

Neutron Probe results: mid slope access tube

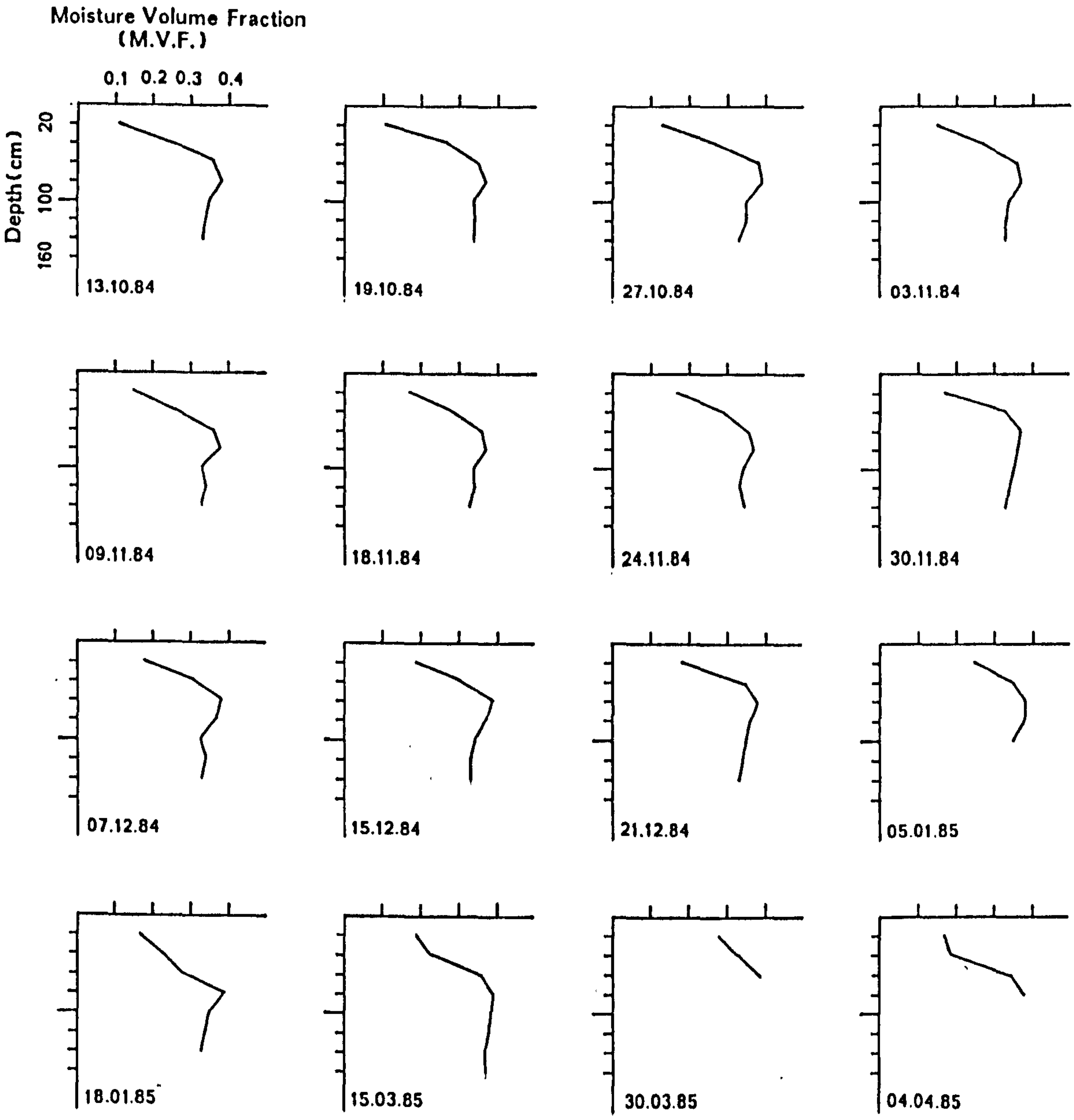



FIGURE 5.30

Neutron Probe results: lower slope access tube

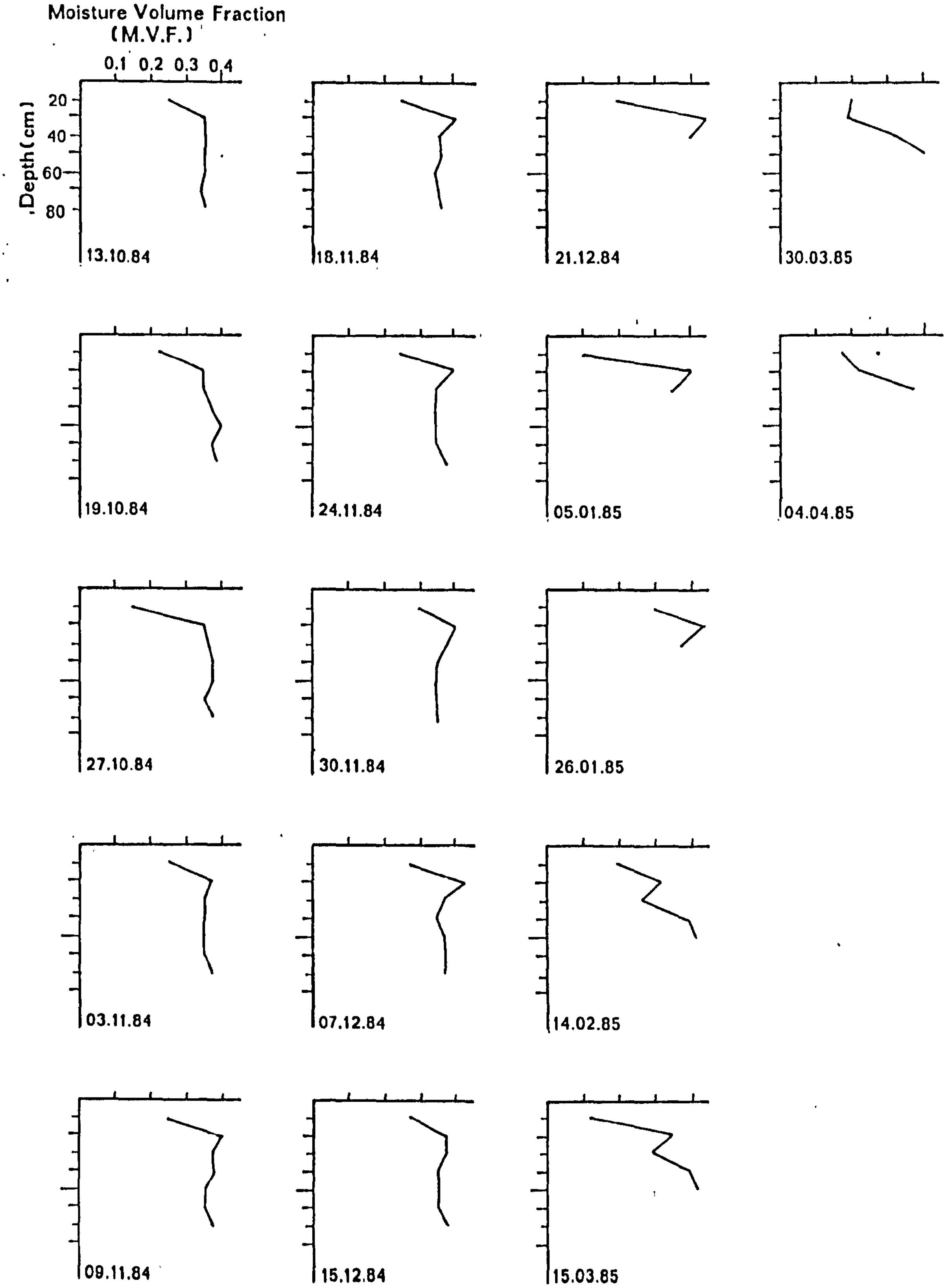



FIGURE 5.31

Neutron Probe results: toe slope access tube present within the sphere of influence of the probe. Towards the end of the recording period noticeably drier surface layers were identified with an increase in the m.v.f. below 30 cm. This reflected lower monthly precipitation totals and a reduced positive hydrological flux.

In the upslope quarter of the mudslide (Figure 5.28) m.v.f. values were similar to those at the top of the slope, with increasing values with depth, greater soil moisture through the whole profile as climatic conditions deteriorated and decreased m.v.f. values towards the end of the winter. The shape of the m.v.f. curve remained reasonably constant throughout the record and the most pronounced change occurred at the ground surface.

M.V.F. values in the lower quarter of the slope (Figure 5.30) were found to be similar to those previously discussed. Further description would be repetitive. Records at the mid-slope (Figure 5.29) and toe slope (Figure 5.31) locations, however, displayed a number of differences. The soil moisture profile present on other parts of the slope was not found here. Recorded values at different depths did not appear to have any clear pattern or trend and following an increase in the m.v.f. throughout the profile as climatic conditions deteriorated, the results remained relatively constant.

The m.v.f. generally showed increases in soil moisture towards the toe of the slope, with greater between measurement point differences higher up the slope than towards the toe. Distortion of the access tubes occurred later in the record towards the toe of the slope, suggesting that seasonal mudslide movements were surge or pulse events moving down the slope. Tube distortion took place after 10 weeks at the slope crest, 14 weeks at the centre of the mudslide and 15 weeks at the toe. This phenomenon may be similar to the unsteady, non-uniform landslide motion, in which localised perturbations propagate slowly downslope as kinematic waves, recently reported by Iverson (1985).

Much detail was thus available on changes in soil moisture within the mudslide both spatially and temporally. It was also hoped that this technique would be able to identify the location of the slip surface. This was not possible since the resolution of the probe was not fine enough to accurately locate such a narrow boundary.

### 5.6.4 Porewater pressure

Changes in porewater pressure recorded using Casagrande piezometers are presented as water levels above the base of the standpipe (Figures 5.32 - 5.35). The recorded head of water fell to zero at all but two of the standpipes during the summer months, while the toe slope piezometers at Stair Hole and Worbarrow maintained artesian pressures throughout the year. The maximum head of water was recorded in January and February, with a rapid increase preceding these peaks and a corresponding decrease afterwards. These changes accompanied alterations in precipitation and the hydrological flux but with an associated time lag of approximately 8 to 10 weeks. At Stair Hole (Figure 5.33), Mupe Bay (Figure 5.34) and Worbarrow Bay (Figure 5.35) peaks occurred both in January and February while at Durdle Door maximum values were dominant in January. This suggests a more rapid response of the phreatic surface at the latter site to

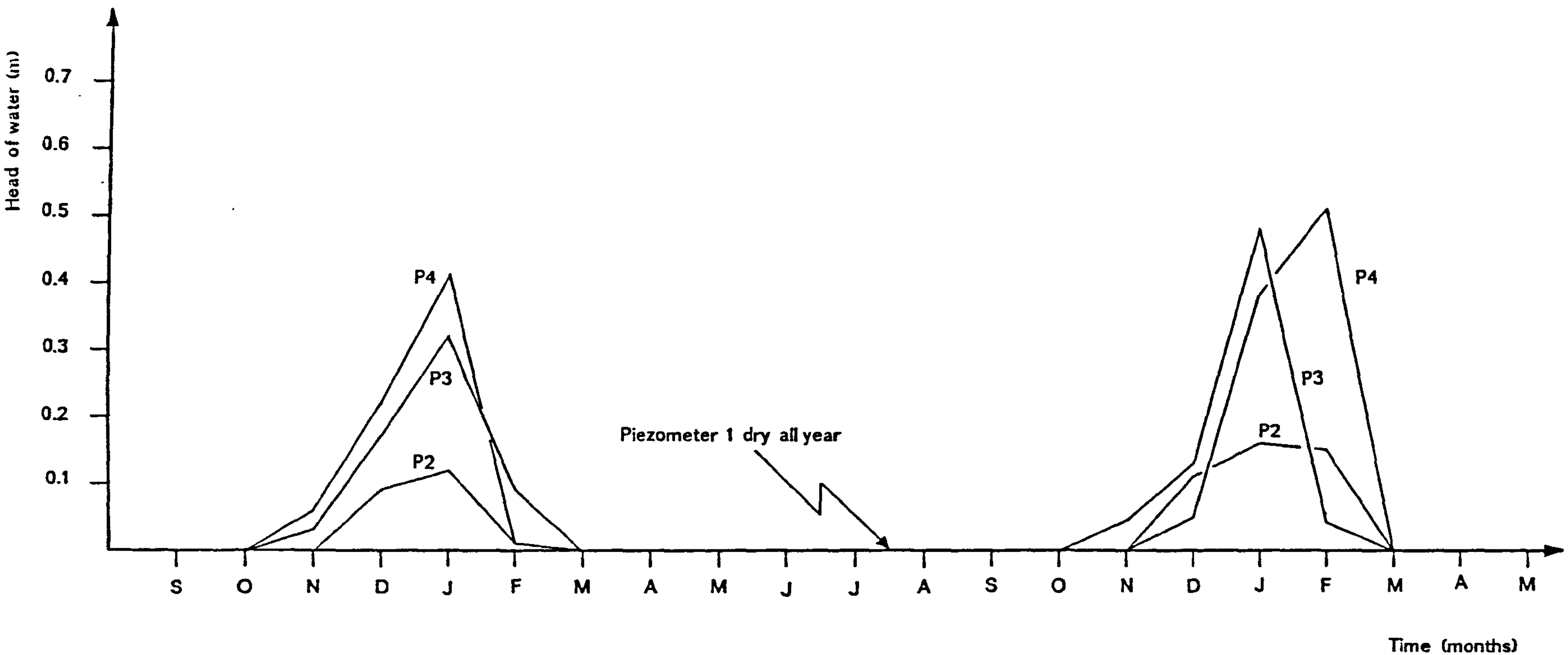



FIGURE 5.32

Phreatic surface recorded on the mudslide at Durdle Door

Location of piezometers:

P1: top slope
P2: upper slope
P3: lower slope
P4: toe slope

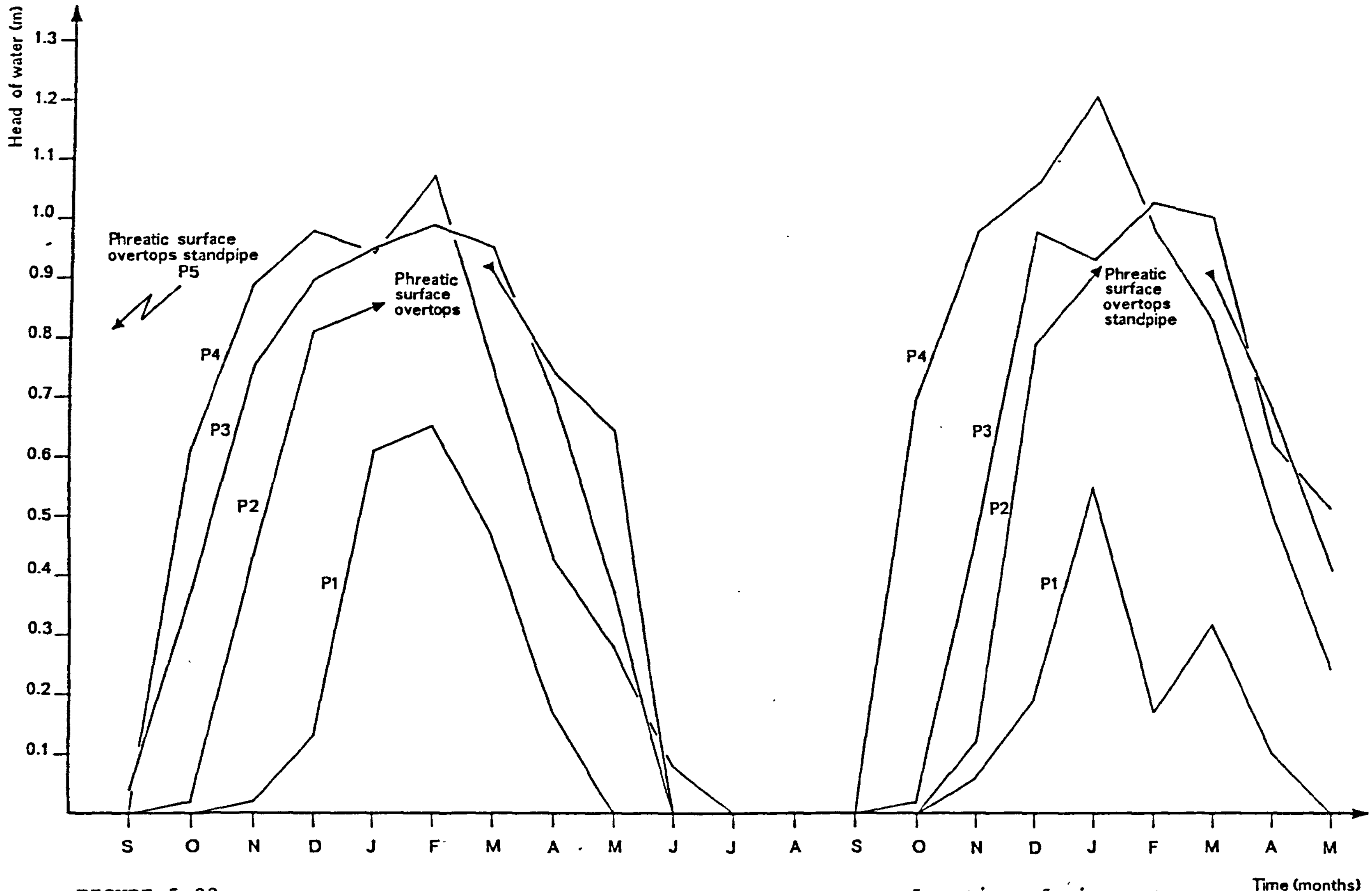



FIGURE 5.33

Phreatic surface recorded on the mudslide at Stairir Hole

Location of piezometers:

P1: top slope
P2: upper slope
P3: mid slope
P4: lower slope
P5: toe slope

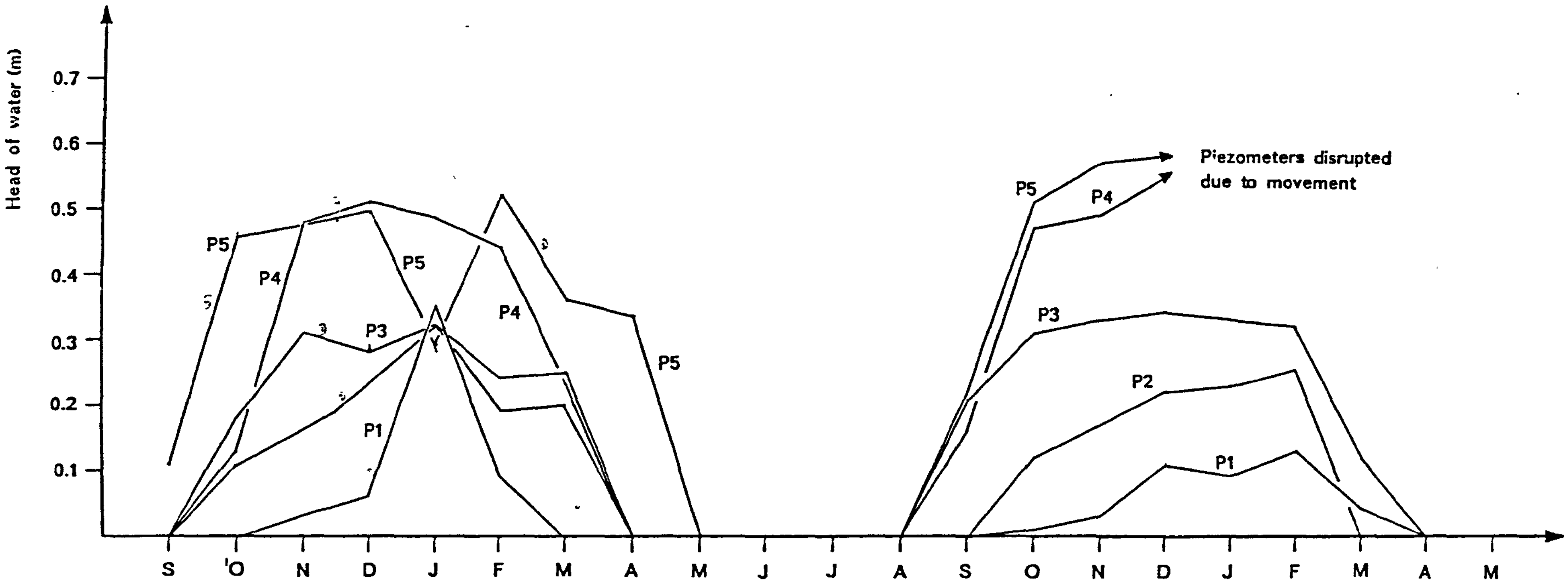



FIGURE 5.34

Phreatic surface recorded on the mudslide at Mupe Bay

Location of piezometers:

P1: top slope
P2: upper slope
P3: mid slope
P4: lower slope
P5: toe slope

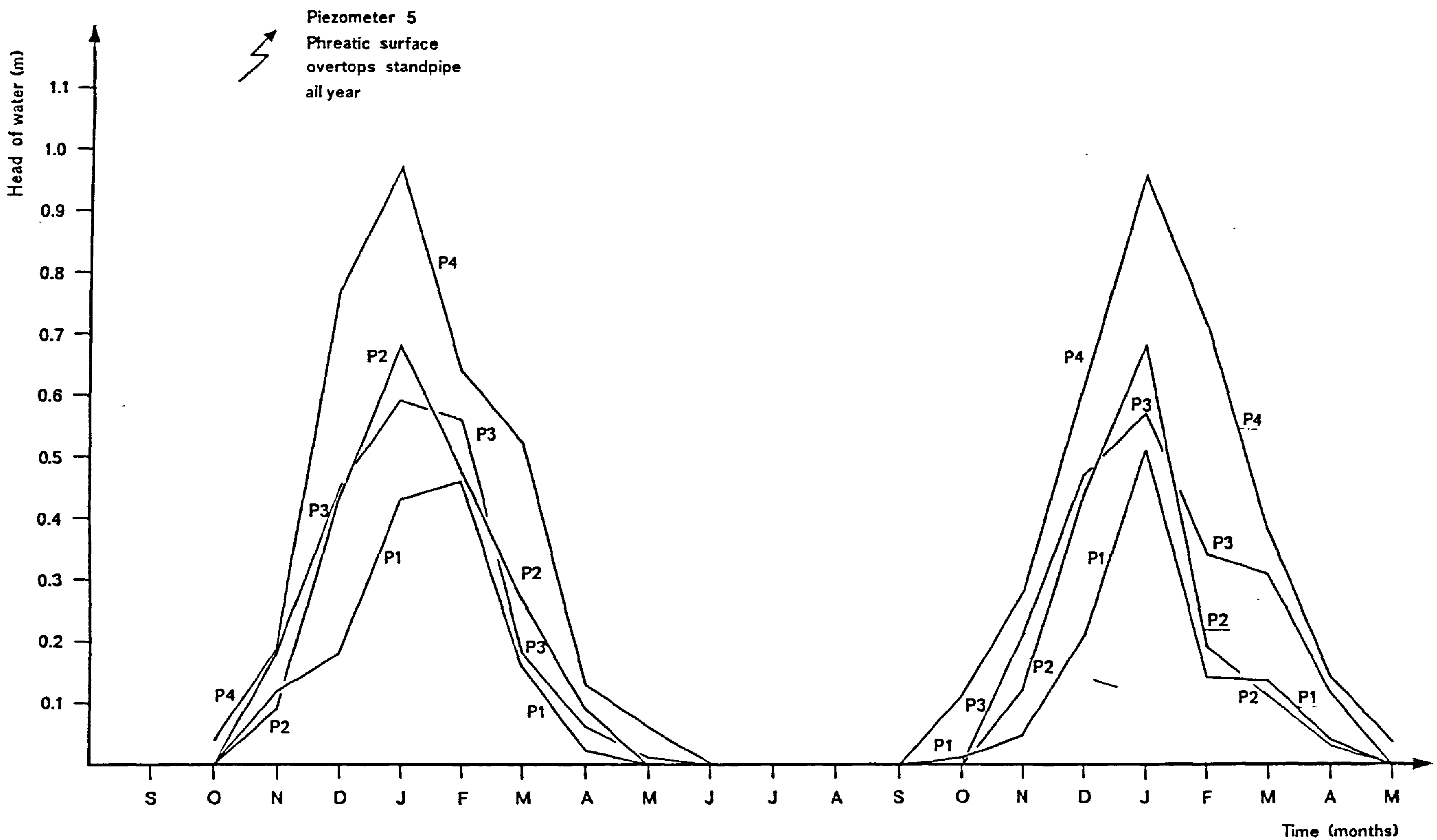



FIGURE 5.35

Phreatic surface recorded on the mudslide at Worbarrow Bay

Location of piezometers:

P1: top slope
P2: up slope
P3: mid slope
P4: lower slope
P5: toe slope

environmental conditions. This is probably due to the geotechnical characteristics of the material (see chapter IV). Particle size is considerably coarser at Durdle Door, permitting faster percolation of water through the mudslide matrix than at other sites. The time at which porewater pressures peaked, the date at which the phreatic surface fell to zero and also started rising again, varied between the mudslides.

A number of site specific characteristics can be detailed. At Durdle Door during both 1983 and 1984, porewater pressures began to rise in October and November. There is a clear time lapse between the seasonal rise in precipitation during July and August and a rising phreatic surface. The increase was later at Durdle Door than at other locations. Throughout the record all the standpipes at this site were dry by March, two months before installations at other sites. This will again be a consequence of the sandy matrix at Durdle Door permitting faster drainage and dissipation of porewater pressures when compared with the other mudslides. It can also be used to explain the lower maximum porewater pressures at Durdle Door. At the slope toe, water levels rose to no more than 0.55 m and the top slope standpipe remained dry all year.

Porewater pressures at Mupe Bay (Figure 5.34) bear closest similarity to those at Durdle Door. Initial increases in the phreatic surface followed expected trends in 1983 but in 1984 the rise commenced in August, one month earlier than at other sites. This probably reflects the small size of this feature and in particular its shallow depth. Other recorded values at this site follow expected trends except for the toe slope standpipe (P5) in January 1984, which registered a distinct drop in values, probably due to the increasingly rapid movements recorded at this time of year and the associated development of tension cracks and shears across the mudslide surface. Where these occurred close to a standpipe,the phreatic surface was bound to fall but as displacement rates declined fissures closed up, accounting for the return of dipmeter readings to normal seasonal values.

Records at Stair Hole (Figure 5.33) and Worbarrow Bay (Figure 5.35) conform to expected seasonal trends. At each site artesian phreatic conditions persisted at the toe slope throughout the year. Water levels also overtopped the upper standpipe (P2) at Stair Hole each year in January, February and March. Other trends were the same as for the previously discussed sites and consequently further discussion here would not bring to light any other salient details. A number of further general points do warrant discussion however.

At any one time, porewater pressures were generally lower towards the crest of the mudslide but there are a number of noticeable exceptions. At Mupe Bay during January 1984, the phreatic surface at the slope crest (P1) exceeded levels lower down the mudslide. Although this is due to abnormally low values in one instance (P5) the comparison of 1983/1984 data with the curves of 1984/1985 suggest that the porewater pressure at the foot of the mudslide was uncharacteristically high in January 1984. A similar anomoly occurred at Worbarrow Bay where the head of water was greater in the centre of the mudslide (P2) than at lower points on the slope (P3). It is suggested that these anomolies are due to localised influences. The phreatic surface at the rear of the mudslide at Worbarrow, for example, may respond much more rapidly to rainfall events,since the slip surface is much closer to the ground surface.

Useful comparison can be made between piezometers installed at similar points on different mudslides. There was an increase in the general overall level of the phreatic surface from Durdle Door to Mupe Bay, Worbarrow Bay and Stair Hole. This reflects the morphology of each mudslide with higher porewater pressures reflecting larger scale features. It does not follow, however, that those slopes which displayed the highest phreatic surface would also be likely to experience the greatest movements since ground conditions, geotechnical characteristics and mudslide form dictate when and how much movement will occur.

The rising and falling limbs of the porewater pressure curves were different. Increases in the head of water at different points on one slope occurred with broadly similar magnitude. Falling phreatic curves were more complex and porewater pressures did not drop uniformly along the slide. This suggests that the controls over rising and falling pressures either vary or operate at different rates.

Periods of zero record in the summer months are likely to be important to annual stability conditions, with suctions rather than pressures exerting an influence over the slip surface. Recent studies (Craig, 1979; Hutchinson & Gostelow, 1979) suggest that suctions across a slip surface can exert considerable influence over subsequent mass movements, increasing the time lag between rises in precipitation, the generation of positive porewater pressures and the commencement of movement.

Finally, important additional details emerge from the Worbarrow Bay record, if a plot of the results obtained from the standpipes is compared with weekly data obtained from the diaphragm piezometers (Fig.5.36). Diaphragm recordings of the phreatic surface at the toe of the mudslide show water levels rose to 1.8 m, 0.7 m above the top of the adjacent standpipe (P5). The toe slope record shows the phreatic surface gradually increased to maximum values and immediately declined thereafter. The toe and top slope records tend to mirror each other in their seasonal trend but with some clear weekly fluctuations. Toe slope phreatic levels remained high all year despite a seasonal change. Diaphragm piezometer records at the top of the mudslide reflect the standpipe records but with enhanced detail. Fluctuations in the seasonal rise in values towards early January 1985 reflected short term climatic variations, mudslide movements and backslope loading. General trends were, nevertheless, similar to changes noted from the standpipes suggesting that the cost of installation and maintenance of equipment such as diaphragm piezometers is only justifiable when wishing either to examine detailed short term variations or obtain data of maximum phreatic levels. This is particularly important where movement in surges are analysed, since the response time of standpipes is not sufficiently rapid to record.

### 5.6.5 Mudslide Movements

A general pattern of movements can be identified at all four sites (Figures 5.37 - 5.40). From static conditions at the start of the record movements were initially slow but increased during September, October, November and December 1983. Total displacements during

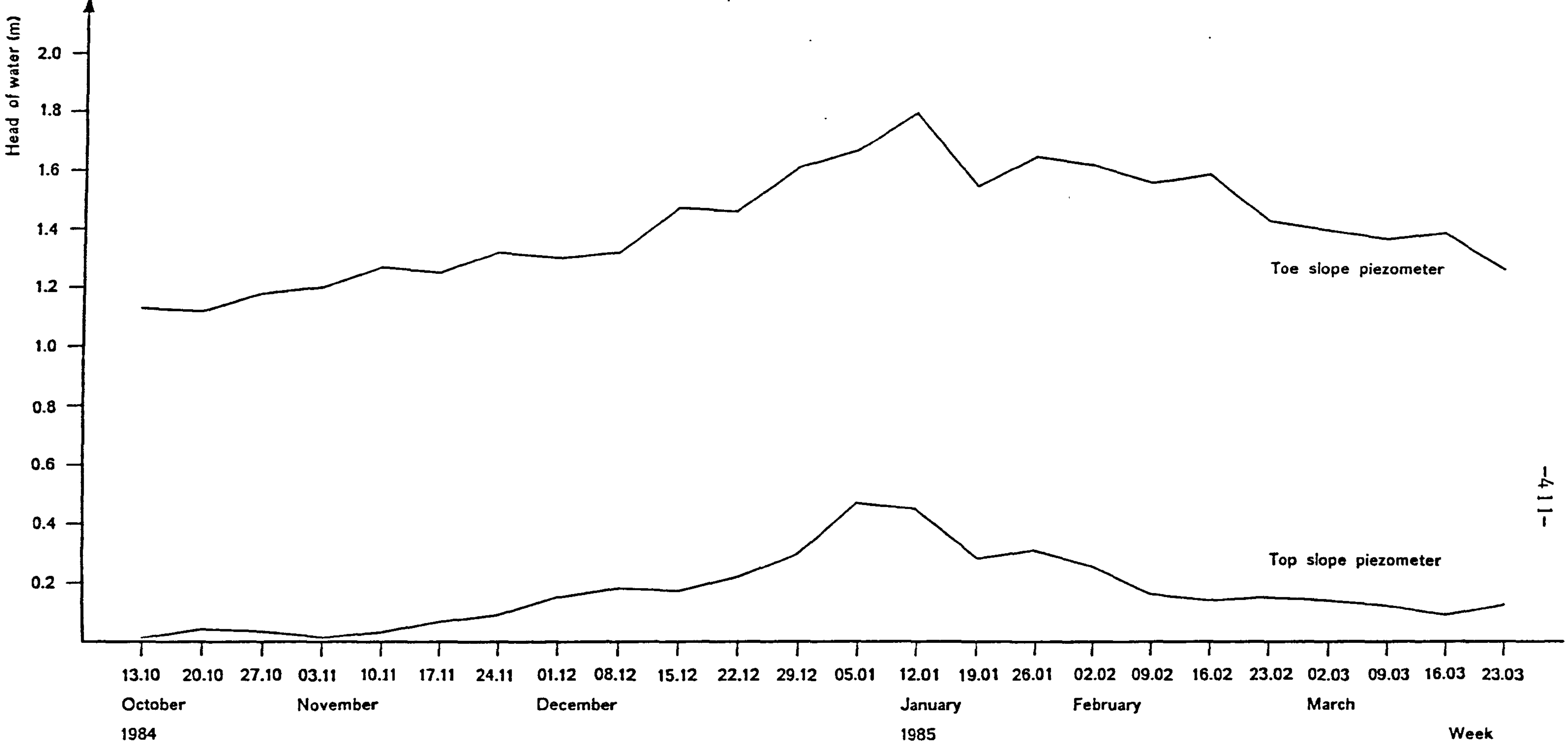





FIGURE 5.36

Phreatic surface recorded on the mudslides at Worbarrow Bay using pressure transducer diaphragm piezometers

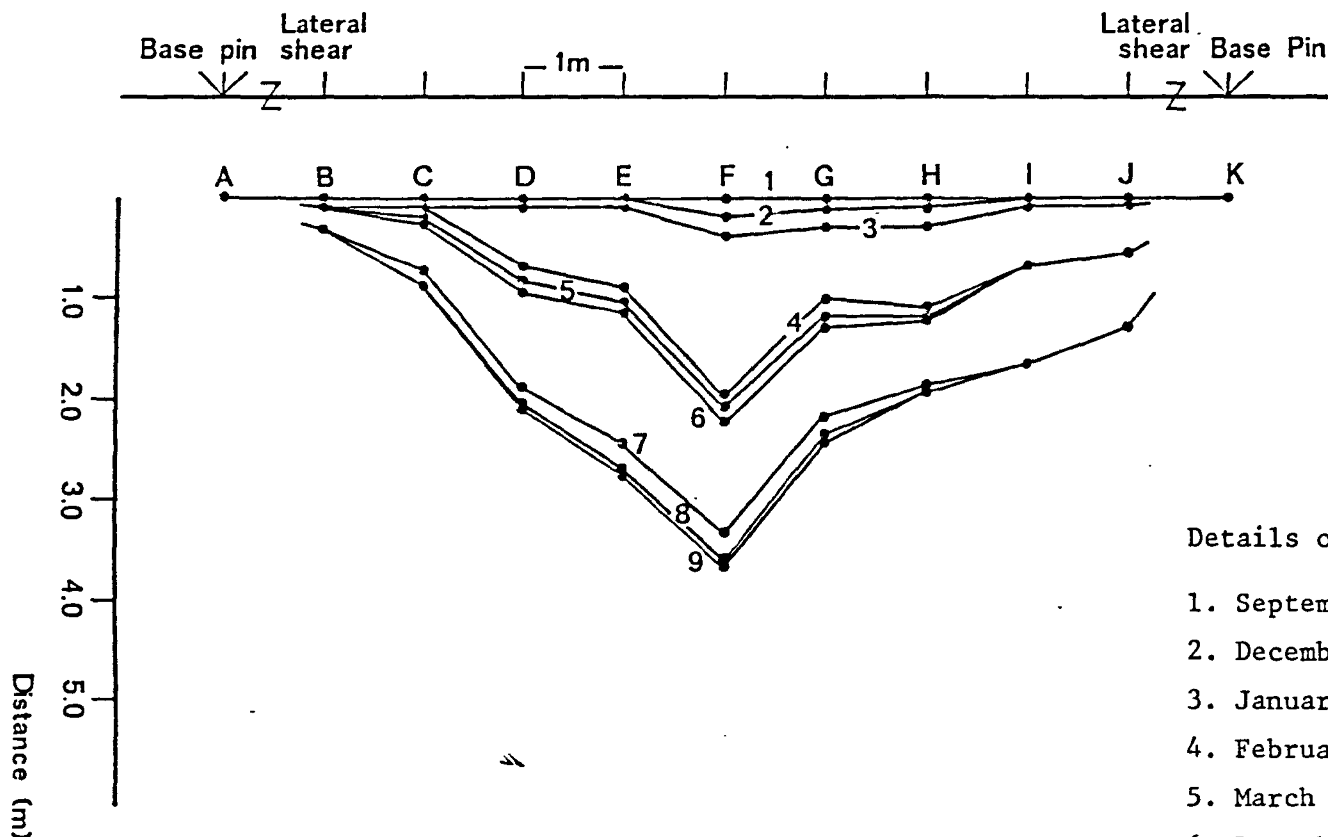



FIGURE 5.37

Pin movements on the mudslide at Durdle Door

Details of movements

1. September - November 1983
2. December 1983
3. January 1984
4. February 1984
5. March - November 1984
6. December 1984
7. January 1985
8. February 1985
9. March - May 1985

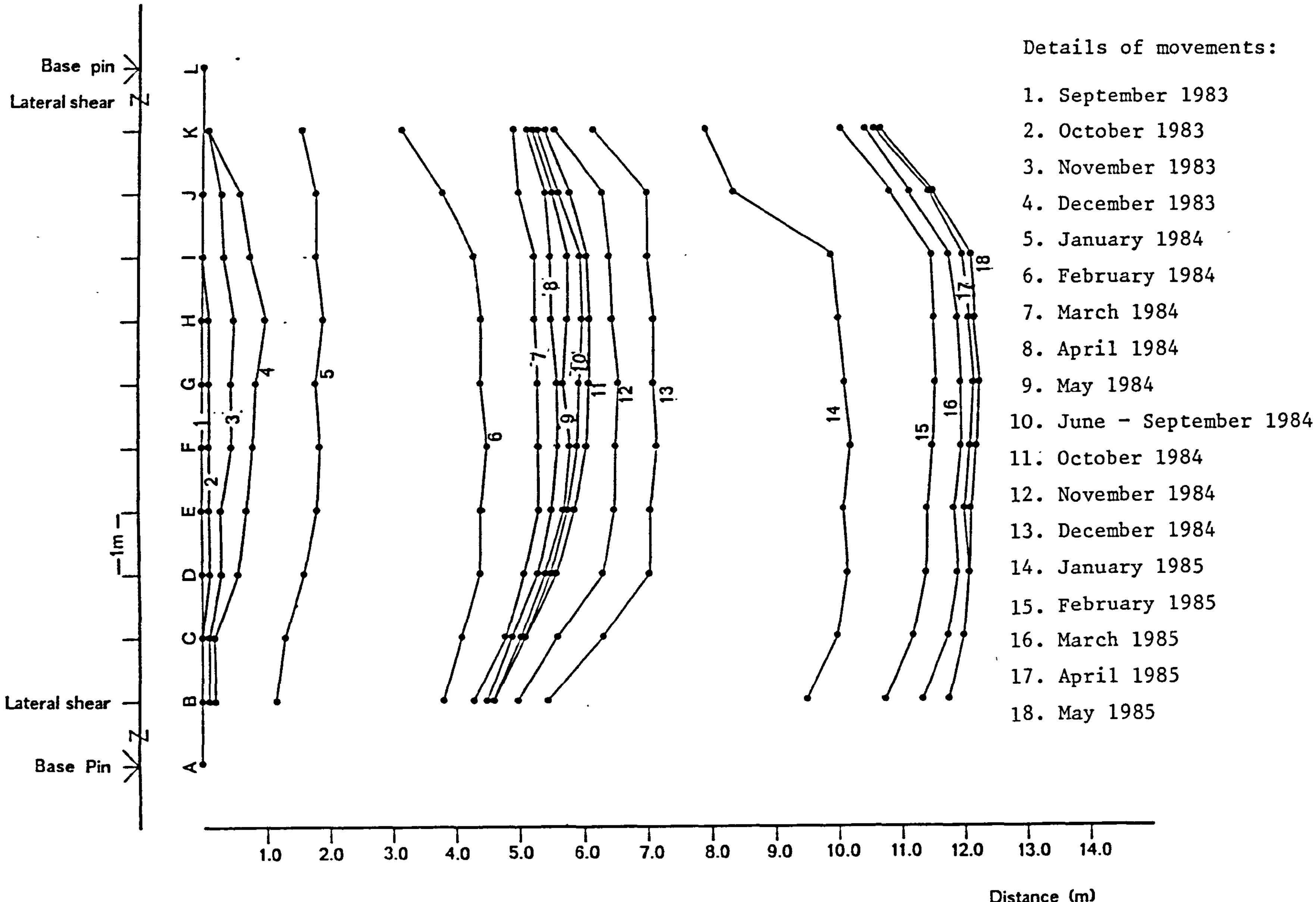



FIGURE 5.38

Pin movements on the mudslide at Stair Hole

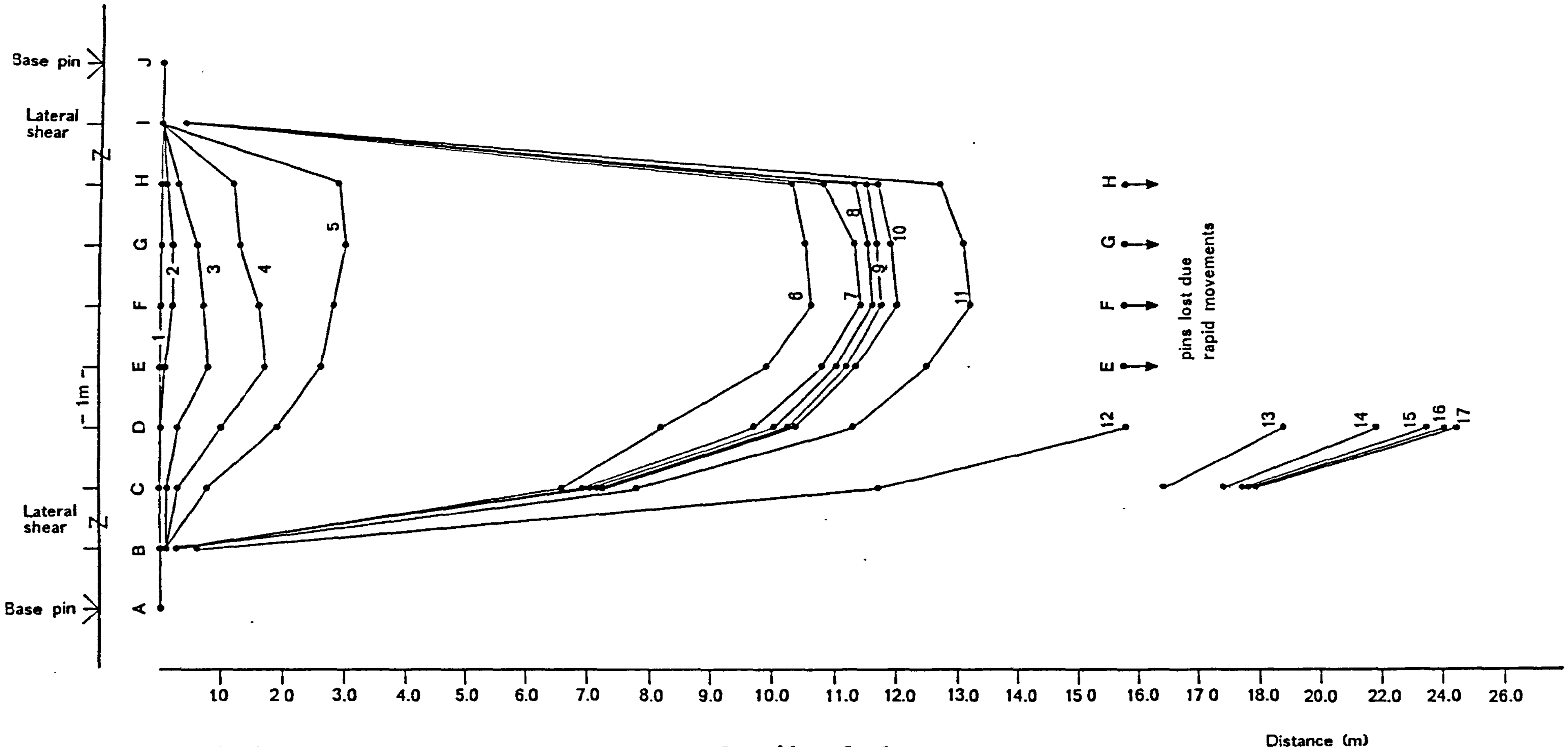
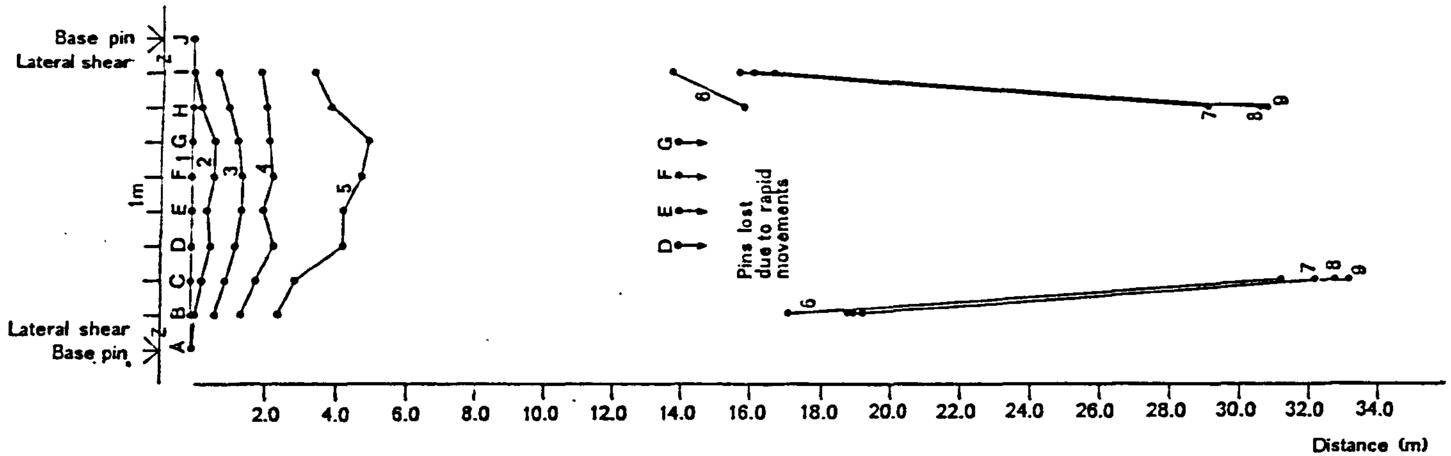



FIGURE 5.39

Pin movements on the mudslide at Mupe Bay

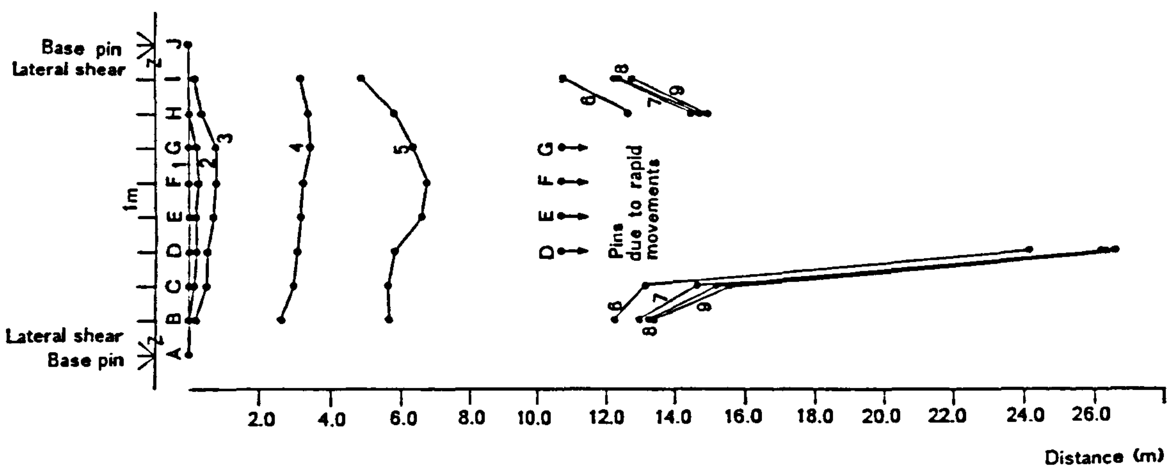
Details of slope movements:

1. September 1983
2. October 1983
3. November 1983
4. December 1983
5. January 1984
6. February 1984
7. March 1984
8. April 1984
9. May - Sept. 1984
10. October 1984
11. November 1984
12. December 1984
13. January 1985
14. February 1985
15. March 1985
16. April 1985
17. May 1985




Details of slope movements

1. September 1984
2. October 1984
3. November 1984
4. December 1984
5. January 1985
6. February 1985
7. March 1985
8. April 1985
9. May 1985




Details of slope movements

1. September 1983
2. October 1983
3. November 1983
4. December 1983
5. January 1984
6. February 1984
7. March 1984
8. April 1984
9. May 1984

FIGURE 5.40

Pin movements on the mudslide at Worbarrow Bay these months varied between sites. At Durdle Door there was no recorded change until December, while at other sites increasing displacement occurred from September onwards. Total rates varied spatially. By November 1983, for example, maximum displacements of 1.0 m, 2.0 m and 4.0 m were recorded at Stair Hole, Mupe Bay and Worbarrow respectively. Movements therefore had both spatial and temporal components.

All sites displayed their most rapid change during January and February for both the 1983/1984 and 1984/1985 seasons, although again there was a difference in the degree of alteration between mudslides. The record for January and February 1983 indicated slippage of 2.5 m at Durdle Door, 3.0 - 3.5 m at Stair Hole and 7.0 - 8.0 m at Mupe. At Worbarrow Bay the movements were so large that the four pins in the centre of the mudslide were lost. Those that remained indicated minimum shifts of 16.0 - 18.0 m. Following these surges, monthly displacements decrease and become insignificant by June 1984. The duration of this period of little or no movement was the same at Stair Hole, Mupe Bay and Worbarrow but longer at Durdle Door, lasting from April to December. Recordings during the 1984/1985 season reflected the patterns of the previous year with a general increase in movement during the autumn and early winter, a sudden surge and an ensuing decrease in displacements. The magnitude of the surges in the 1984/1985 winter was greater than for the previous year, highlighted by the evacuation of the central part of the mudslide at both Mupe and Worbarrow Bays. Two alternative hypotheses can be suggested for this. Firstly, an additional factor might enhance movements reflected by the particularly large displacements. Secondly, critical thresholds may exist for important variables governing the stability of the mudslides. Where these are exceeded, particularly large movements can be expected for only small changes in controlling parameters. Individual site records can be used as cases in point. At one extreme these thresholds were crossed during both years at Worbarrow Bay, resulting in large displacements. At Durdle Door and Stair Hole on the other hand, the limits were not exceeded. Although seasonal changes in movements were identified, particularly large surges did not occur. At Mupe Bay during the 1983/1984 season the variables governing movement must have come close to critical values but without crossing threshold values. During the 1984/1985 winter however, these critical levels were exceeded and consequently a surge occurred.

Differences were also found in the detailed nature of displacements between sites. Durdle Door results displayed side friction effects, with movement distorting the pins installed across the slope from a straight line to a parabola. A similar but less pronounced pattern resulted at Stair Hole. These differential movements seemed to occur mostly during rapid changes in the winter months. At Stair Hole, for example, between December 1984 and January 1985 the movements accounted for most of the spatial separation between pins I, J and K noted at the end of the season. At Worbarrow and Mupe Bays, although general monthly variations in the amount of movement were the same as at other sites, those pins in the centre of the mudslide mass moved downslope en-masse.

While separate examination of the recorded movements and the other previously discussed data sets is important, it is equally necessary to make comparisons between the different records. Such a synthesis will assist explanations of the causes of movements, their magnitudes and when they occurred.

### 5.6.6 Discussion

Associations can be identified between the variables described above. Salient details identified for discussion draw the monitored parameters together in a sequence of cause-effect events. A rise in precipitation totals and an associated increase in the hydrological flux had a direct control on soil moisture and porewater pressure. These in turn exerted an influence on the detachment of material from the rear scar and hence the rate of loading at the head of the slope.

As precipitation totals increased, so other climatic parameters including air temperature and relative humidity changed, affecting the water flux which rose throughout the autumn and early winter. During September, October and the beginning of November the amount of material which became detached from the rear scar remained small and can be attributed to subaerial weathering. The change which suddenly occurred in the middle of November, with large blocks of material falling onto the rear of the mudslide followed,the deterioration in climate and increases in the water budget. Rising soil moisture and perhaps progressive softening resulting in reduced material strength,led to the detachment of the most unstable material. This will, in turn, have caused an increase in the potential instability of the adjacent areas of the rear scar. Once major detachment has occurred in any one year,the freshly exposed scar remains stable again until the following winter. Separation therefore relied on gradual changes throughout the whole of the year reducing the stability of the overall mass, as well as seasonal controls. Those components which change gradually through time are not greatly affected until the material is exposed at the crest of the slope.

All Neutron Probe results indicated an increase in soil moisture throughout the autumn and winter. Soil moisture appeared to rise with a high positive water balance, remaining constant rather than experiencing decline when the positive water balance was small. At the same time porewater pressures rose. A considerable amount of moisture was required following the summer months before the porewater pressures were affected however. Any water surplus before the phreatic surface started to rise was probably taken up as soil moisture. A lag can be identified between maximum precipitation values and peaks in porewater pressure. If it is assumed that phreatic surface maxima reflect rainfall peaks, the time taken for piezometric response to climatic conditions can be identified. This differed both between and within sites and also from season to season, generally occurring in either January or February. The lag between water input and piezometric response appeared to decrease as precipitation totals rose towards peak values. At these sites a mean time of 8 - 10 weeks occurred between precipitation and porewater pressure peaks.

The rate of rise to peak porewater pressure values varied from site to site. At Durdle Door and Worbarrow Bay increases were gradual at first, subsequently rising rapidly to maximum levels. In contrast at Mupe Bay and Stair Hole, porewater pressure increases were relatively constant throughout the whole season. Rising phreatic curves showed that the response to increased precipitation and soil moisture took longer than the time required for the dissipation of equal heads of water. This is probably because increases in the level of the phreatic surface relate to changes in the water balance more than falling porewater pressures which are subject to site-oriented, within slope controls.

An examination of the relationship between these parameters and slope movements can now be made. This is best achieved by initially considering the results at each of the four sites. At Durdle Door peak porewater pressures were recorded during January in the 1983/1984 season. Maximum mudslide movements occurred between January and February. The rise in porewater pressure was initially slow compared to other sites, probably accounting for the delay to measureable displacements until December and only a small seasonal surge between January and February. This trend was further highlighted during the winter of 1984/85 when very little movement occurred before the January surge. The largest monthly movements at this site were recorded before peak porewater pressures at one of the piezometers (P4). This suggests that either maximum porewater pressure values were not necessary along the whole slope to initate movement or that the movement took place during higher phreatic levels than are recorded here, these occuring between sampling intervals.

At Stair Hole maximum movements were recorded between January, February and December 1984 and January 1985 with maximum porewater pressure values being recorded at approximately the same time. Again it is possible that the maximum phreatic surface remained undetected, occurring somewhere between the remeasurements in December and January. Movement rates at this site did not decline as rapidly as those at Durdle Door and the phreatic surface had to drop considerably before displacements became insignificant between individual months.

The associations between these parameters at Mupe and Worbarrow Bays were similar in many respects. At both sites maximum porewater pressures in the first season occurred in January 1984, with peak movements in February. As porewater pressures rose there was a curvilinear increase in movement rates until the point was reached at which major displacements occurred. Following the surge there was an exponential decrease in movement month by month, accompanying the falling phreatic surface. At Mupe Bay identifiable movements were noted as soon as the phreatic surface began to increase, indicating rapid response to small changes in the head of water. Results recorded during the 1984/85 season mirror those of 1983/1984, barring a small number of exceptions. At Mupe, as previously discussed, a particularly large surge occurred in November 1984 accompanying a more rapid rise in the phreatic surface than in the previous year and reflecting above average precipitation totals.

Despite the conclusions which can be drawn by comparing these data, difficulties of interpretation remain which are associated with the temporal sampling framework. This is inherent to all studies of this nature. Although broad trends were identified, it was impossible to examine parameters in detail and at points of specific importance in the complete record, such as immediately before movements. Since the most significant association is that of porewater pressure and movement, it was decided to conduct a detailed examination of these two variables using a particularly fine temporal sampling base.

## 5.7 RESULTS FROM DETAILED MONITORING AT WORBARROW BAY

The results obtained using data logging techniques at Worbarrow Bay provide new details about patterns of mudslide movement and associated changes in porewater pressure which add considerably to the information provided by Hutchinson et al., (1974) and Craig (1979). Due to the size of this data set, which exceeds 480,000 recordings of slope movement and 120,000 readings of porewater pressure, it is clearly impossible to present all the results. Data presented so far indicates that, over large periods of time, no significant change occurs in these variables. Conversely, a number of significant events can be identified from the monthly records, but their precise details remain unexamined due to the temporal sampling framework. From the complete record collected using the data loggers selected periods of important change were studied using the results of the previous year as a guide. Three types of movement event can be identified:

(i) Small individual spatial movements which are grouped temporally. For the purposes of this discussion these will be termed 'multiple' movements.

(ii) Gradual spatial movements which occur within defineable temporal limits. These will be termed 'graded' movements.

(iii) Rapid movements occurring within a short space of time and involving large spatial displacements. These will be referred to as 'surge' movements.

The three types of event occurred with different frequency and at different points in the overall record. All were separated by periods of no displacement. One particular example of each type of movement will be presented as a case study. Details of the total number of each of the events will be given and other relevent points, such as the time of occurrence and the cumulative movement recorded during each episode, will be given. This will provide detail which cannot be obtained from the previously discussed data.

5.7.1 Multiple Movements

A typical example of a multiple movement is presented in Figure 5.41 and Figure 5.42. The plot of downslope pin displacement against time (Figure 5.41) displays a number of characteristics. In 36 hours between 20.00 on 30th November 1984 and 08.00 on 21st December 1984 a total downslope movement of 8.0 cm was recorded in the centre of the mudslide. This accounted for approximately 15% of the total monthly movement recorded by pin resurvey. This detail was not available from monthly survey data, explaining why this type of movement has not been identified by previous studies, although it has similarities to the slip movements of Craig (1979). Each of the pin movement records during this time showed a similar trend. Individual displacements of no more than 1.0 cm, in all but one exception, were separated by time intervals of varied duration. This resulted in 'jerky' movements as the mudslide material moved

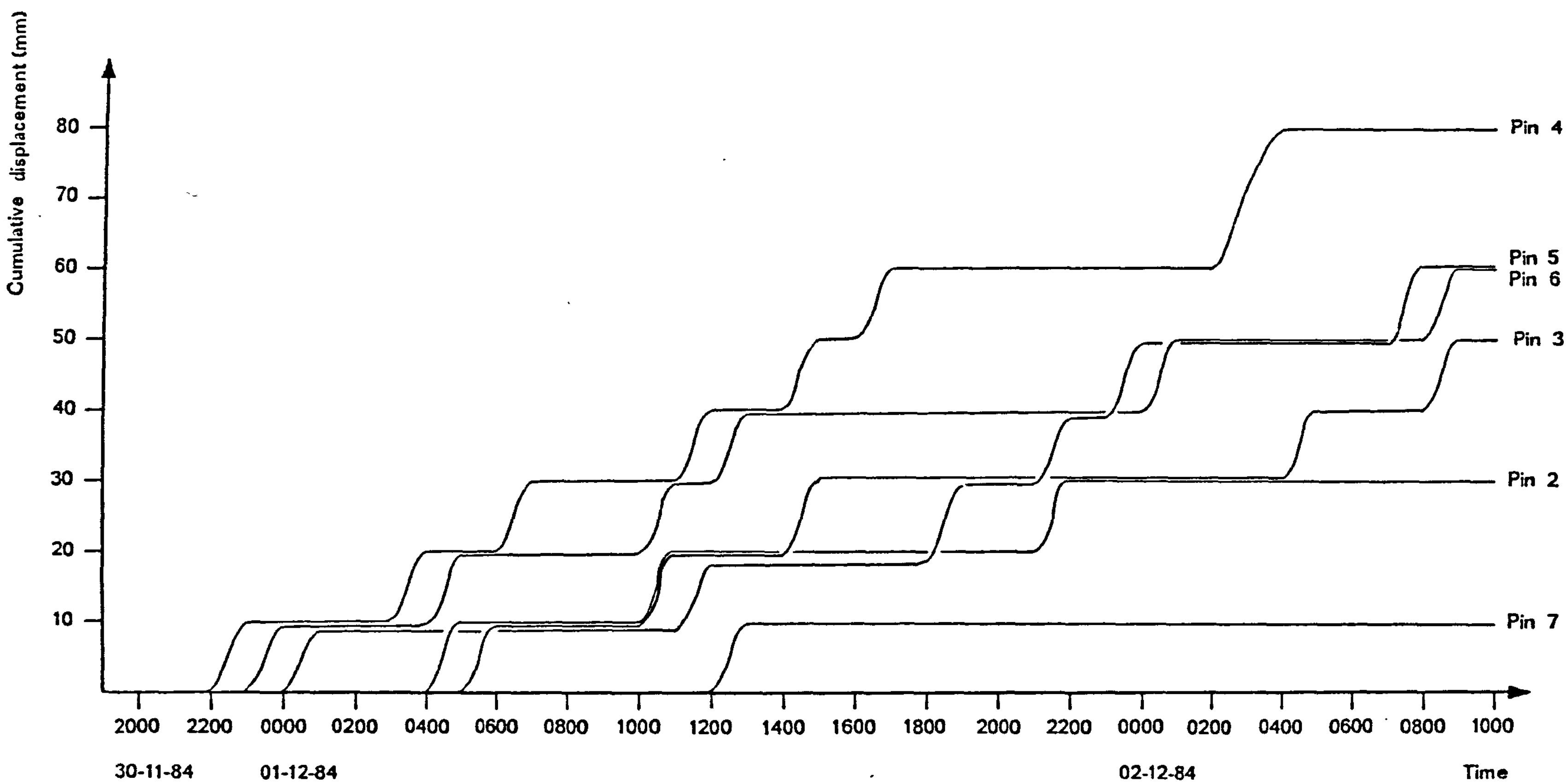



FIGURE 5.41

Example of 'multiple' movement (stick slip) pin displacements recorded on the mudslide at Worbarrow Bay

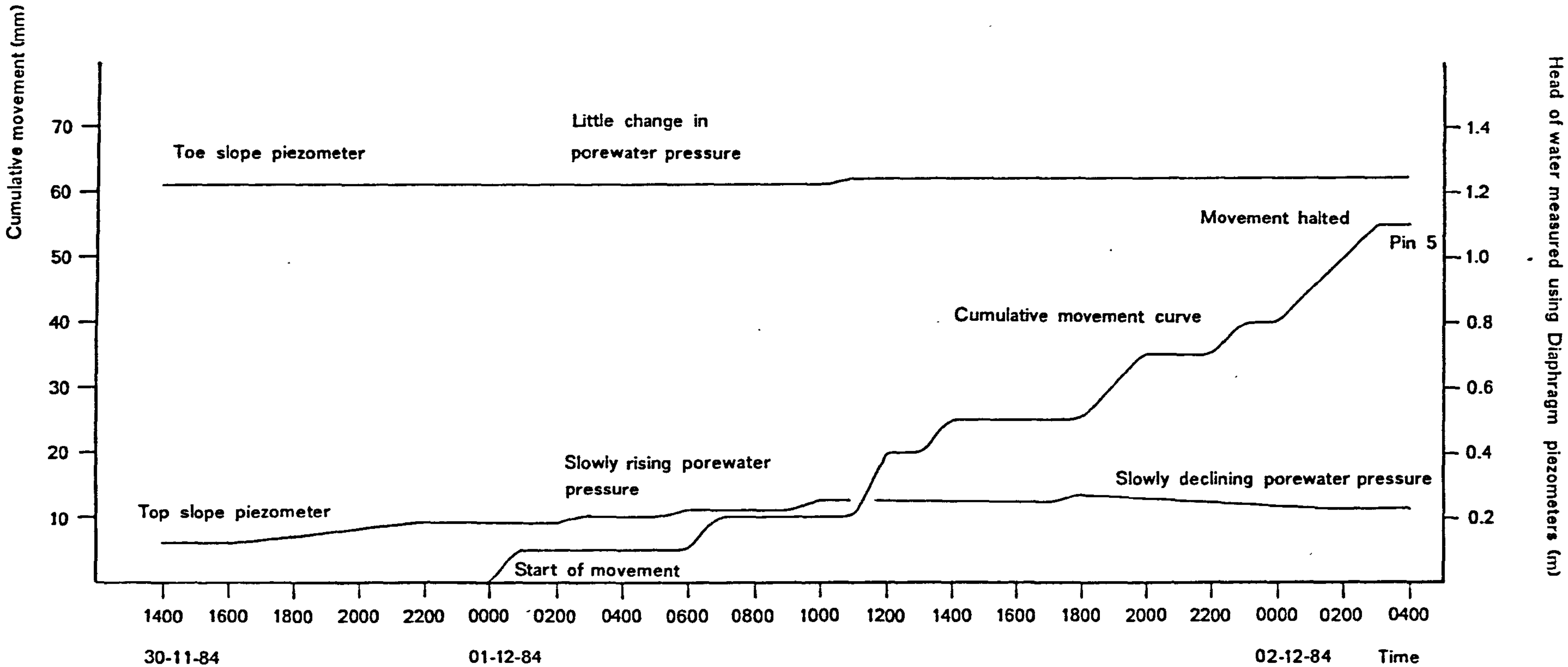





FIGURE 5.42

Relationship between pore water pressure and 'multiple' movement (stick slip) displacements of the central pin recorded on the mudslide at Worbarrow Bay downslope. All pin records followed this pattern but with different specific characteristics to each one, depending on the position of the pin used to record changes on the mudslide.

Two pins installed as controls (Nos 1 & 8) off the main mudslide showed no change. Consequently they do not appear on the record. The pins closest to the lateral shear on the mudslide mass (Nos 2 and 7) demonstrated only small displacement, one (No. 2) moving 3 cm and the other shifting only once during the presented record. The remaining pins moved on a number of occasions in varying degrees but with greater displacements towards the centre of the mudslide. The movements were preceded and followed by periods of no change. More frequent 1 cm movements,occurring in the centre of the mudslide were preceded and followed by similar displacements but with a greater time interval between individual movements. Examining the record of one pin (No. 6) close to the centre of the slope provides a case in point. Slip occurred first at midnight on 30th November 1984. Following this no further advance was recorded until 11.00 the following day when again a slip of 1 cm was followed by immobility until 18.00. Between 18.00 and 23.00 three separate movements of 1 cm occurred. A period of five hours then lapsed with no change until a concluding shift of 1 cm at 07.00 on 1st December 1984. This trend was repeated across the mudslide. On some occasions change occurred at more than one point at the same time, while other events were completely isolated.

Much care is needed in comparing cross slope variations during one such event when movements are so small. Using ten turn rotating potentiometers every rotation of $14.4^{o}$ of arc will change the data logger record by one unit. Differential motion between pins may give rise to a situation where a change of no more than $1^{\circ}$ of arc is required to alter the logged value at one point on the mudslide, while as much as $13^{\circ}$ may be required for an adjacent sensor. Examining small changes on a short time base must therefore be undertaken with care. Although individual slips appeared to be randomly distributed through time, their cumulative effects showed greater displacements in the centre of the moving mass. Even with small movements therefore, a cross-slide differential component was identified within the mudslide.

The movement recorded by one pin in the centre of the mudslide was used to compare displacements with changes in porewater pressure. The top diaphragm piezometer was installed close to the movement sensors, while the second instrument was positioned at the toe of the slope to record the artesian porewater pressures which were unmeasureable using the standpipes. The phreatic surface at the top of the mudslide rose slightly approximately eight hours before the first movement and a gradual increase preceded slip. Once displacement commenced, a small increase in the rate of rise of the phreatic surface occurred. This may well have been due to the movements themselves rather than external influences. A fall in porewater pressure occurred half way through the period of movement. This and the accompanying new angle of repose of the mudslide was reflected in the decreasing movements and eventual stability.

The record for the toe slope piezometer showed little change during this event and the phreatic surface remained relatively constant. The upslope cumulative movement was small and spread over a number of hours. Any effect this had on the phreatic surface lower down the slope was therefore virtually insignificant, with both an elasticity decay effect and a time lag to any likely influence. Movements seldom occurred along the whole mudslide at the same time. Consequently, the effects of small displacements over long periods at one point probably became increasingly less pronounced through space. It is noteworthy, however, that a sudden small rise in porepressure in the toe slope instrument was recorded at 11.00 hours. This may reflect loading of the accumulation lobe from the upslope movements.

These results can be used to examine the accuracy of records of this nature, relative to the frequency of remeasurement. With monthly remeasurements, general trends of increasing and decreasing porewater pressures were identified with data summarising mean conditions. Weekly sampling showed similar trends but other variations not seen in the monthly statistics occurred, such as undulations in the phreatic curve. Such detail became increasingly pronounced as the length of the sample time base was reduced. The record for the top slope diaphragm piezometer presented here highlights this, for example, with recorded fluctuations relating to specific movement events which would have otherwise passed undetected.

Finally, a model for this type of movement can be proposed. A gradual rise in the phreatic surface and increased soil moisture decreases the stability of the mudslide. A number of small displacements occur, each dissipating a small portion of the pore pressures. These movements can be likened to the 'stick-slip' mechanism described by Brunsden (1984) and Craig (1979, 1981). Although each displacement is small, their frequency increases until a combination of dissipation in porewater pressures, a consequence of the movements, and a more stable angle of repose of the mudslide leads to a cessation of movement. Displacements of this nature are almost imperceptible and accompanying changes in the phreatic surface are equally small. One example of this type of movement has been presented from the complete record. A number of such events occurred during monitoring, increasing in frequency as climatic conditions deteriorated and porewater pressures rose (Table 5.13). These stick-slip 'multiple' movements occurred both as discrete events and also as precursors to some of the larger displacements. As mudslide instability increased,movements became larger and a transition occurred from 'multiple' movements to what are termed here 'graded' movements.

### 5.7.2 Graded Movements

A typical example of this type of movement is presented in Figure 5.43 and Figure 5.44. Maximum downslope displacements of 35 cm occurred over a lapsed time of 17 hours. The movements were of a greater distance over a shorter time span than they were for 'multiple' (stick-slip) movements. This type of displacement bears some of the characteristics of standard seasonal movement patterns. From stability at 10.00 on 23rd December 1984,all pins started to move within four hours of each other. From 15.00 to 19.00 displacements were small and in many respects conformed to the characteristics of 'multiple' movement events. However,from 17.00 on 23rd December 1984 to 03.00 on 24th December 1984, movement rates increased through time and reached a point of constant velocity. The displacements in this example finally declined to zero by 04.30. It therefore appears that in some cases individual movements on

TABLE 5.13

Details of 'multiple movement' (stick slip) pin displacements at Worbarrow Bay

| START OF EVENT | | END OF EVENT | | LOGGED DISPLACEMENT (m) BY POTENTIOMETER | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Date | Time | Date | Time | Pot 1 | Pot 2 | Pot 3 | Pot 4 | Pot 5 | Pot 6 | Pot 7 | Pot 8 |
| 15.10.84 | 0445 | 16.10.84 | 1815 | - | 0.02 | 0.02 | 0.01 | 0.01 | 0.02 | 0.02 | - |
| 22.10.84 | 0900 | 24.10.84 | 0445 | - | 0.01 | 0.02 | 0.02 | 0.03 | 0.03 | 0.01 | - |
| 06.11.84 | 0330 | 07.11.84 | 1230 | - | 0.02 | 0.03 | 0.05 | 0.06 | 0.05 | 0.03 | - |
| 30.11.84 | 2200 | 02.12.84 | 0800 | - | 0.03 | 0.05 | 0.08 | 0.06 | 0.06 | 0.02 | - |
| 29.12.84 | 0915 | 31.12.84 | 0730 | - | 0.03 | 0.05 | 0.06 | 0.05 | 0.04 | 0.02 | - |
| 04.01.85 | 1945 | 05.01.85 | 1630 | - | 0.02 | 0.04 | 0.04 | 0.06 | 0.05 | 0.03 | - |

Note: First mudslide surge occurred on 6 January 1985.
Second mudslide surge occurred on 8 January 1985, destroying gantry.

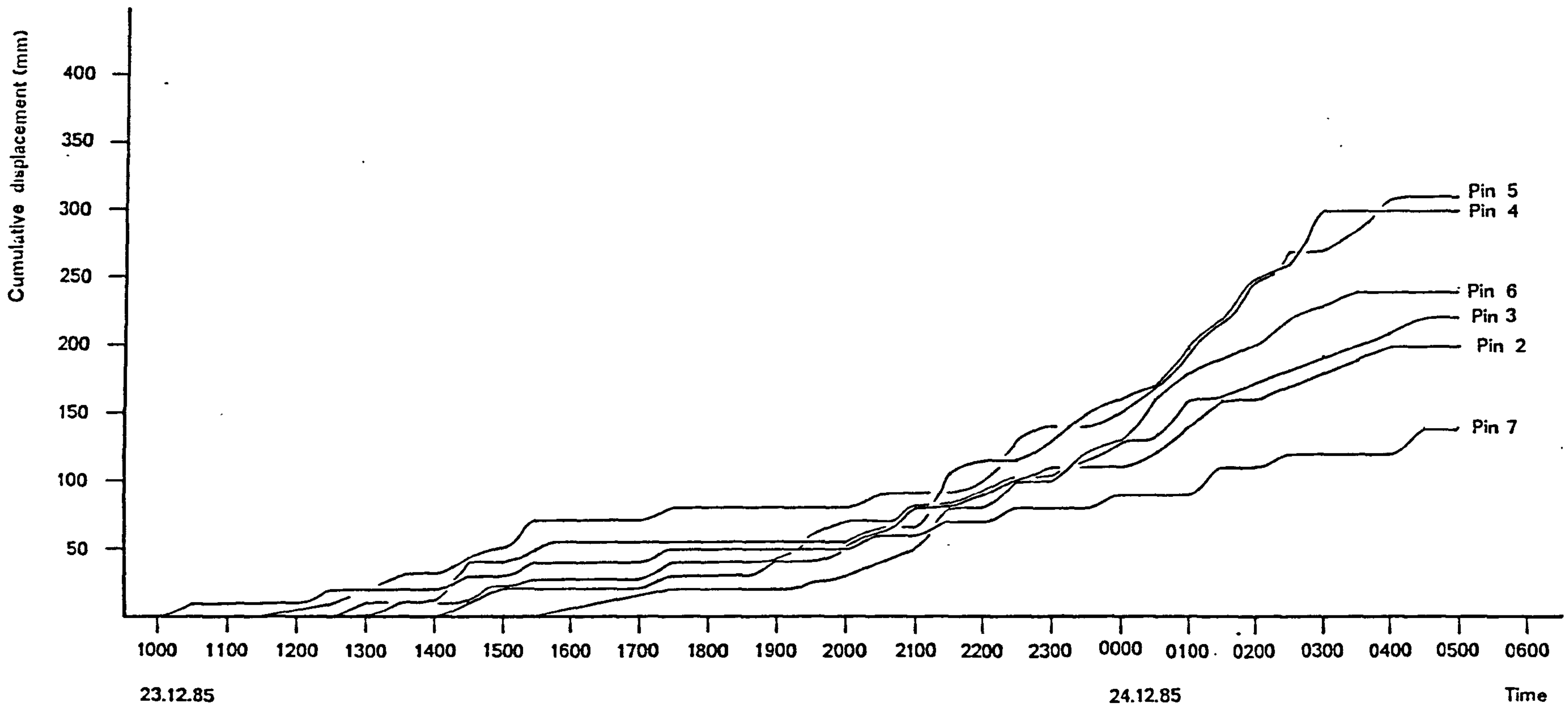



FIGURE 5.43

Example of 'graded' slip pin displacements recorded on the mudslide at Worbarrow Bay

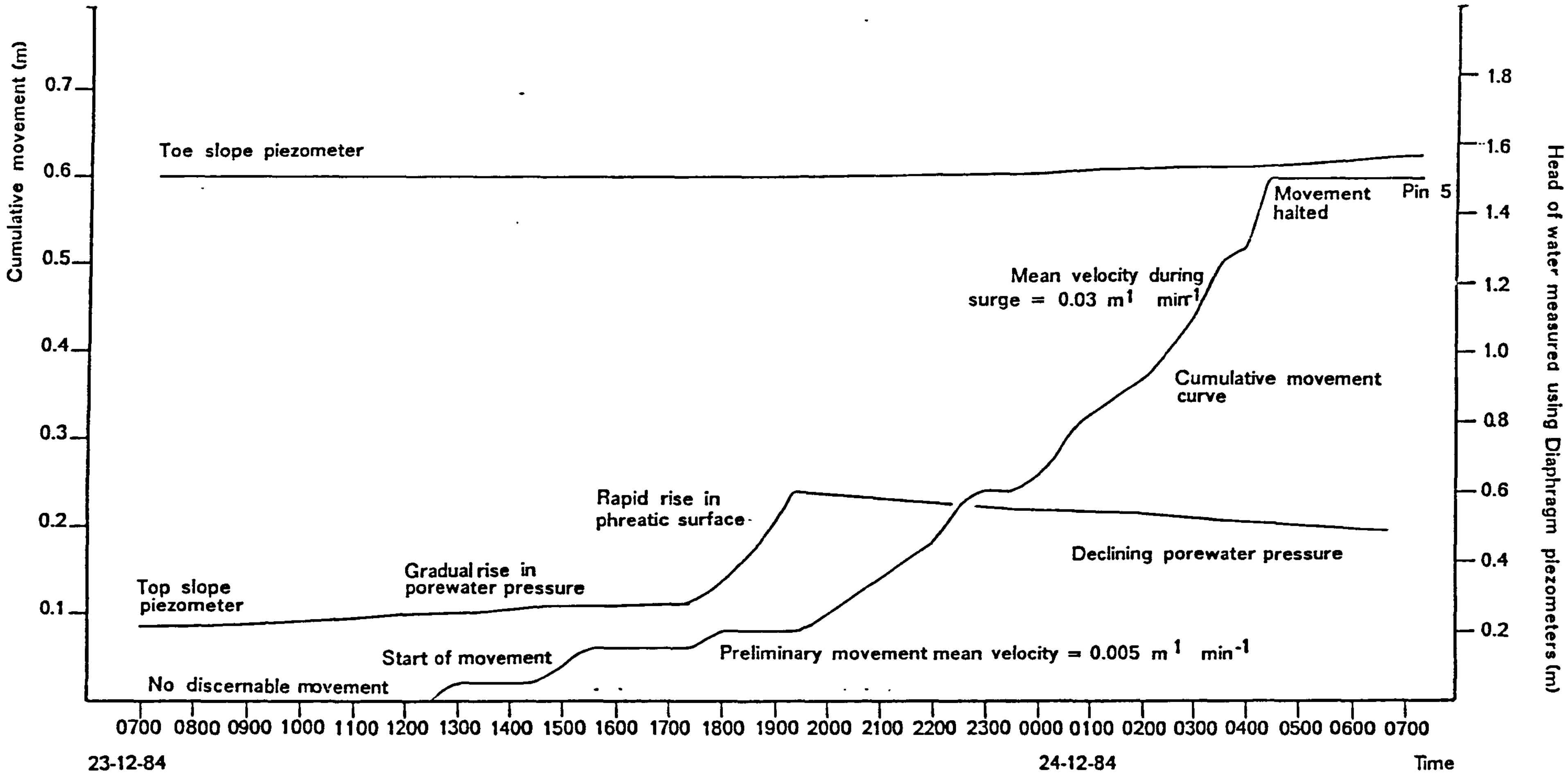



FIGURE 5.44

Relationship between pore water pressure and 'graded' slip displacements of the central pin on the mudslide at Worbarrow Bay mudslides mirror seasonal trends. It is not necessarily either the pins at the centre of the mudslide or those displaying the greatest movements which shift first. In this instance, for example, the material at the edge of the mudslide (pin 7) displayed the smallest cumulative change of 150 mm but started advancing at 1000 on 23rd December 1984 while material in the centre of the mudslide (pin 5) moved more than twice this distance but did not register any change in location until 11.30 on 23rd December. However, as movements continued, relative displacements followed expected trends. During the main period of advance, between 21.00 on 23rd December and 03.00 on 24th December displacement occurred at all points, with increasing magnitudes towards the centre of the mudslide. Downslope advance ceased more abruptly than it started, all pins coming to rest within two hours of each other.

A comparison can be made between these results and the monthly resurvey record for December at this site. In the centre of the mudslide, total monthly movements of 4.0 m were recorded, representing 10% of the overall monthly total recorded by pin resurveys. At the edges of the mudslide, cumulative monthly displacements of 1.0 - 1.5 m occurred. The detailed record shows that at this point some 10 - 16% of this total occurred during the event described above, suggesting that a greater proportion of total monthly movements at the edge of the mudslide occurred during 'graded' movements than during 'multiple' (stick-slip) events.

Further details emerged by comparing the displacement of the central part of the mudslide with porewater values recorded with the diaphragm piezometers (Figure 5.44). The top slope piezometer showed a gradual rise in porewater pressure starting at 07.00 on 23rd December 1984. Initial values reflected mean weekly conditions. The rate of rise of the phreatic surface increased as movement commenced, rising some 0.3 - 0.4 m within the space of two hours. This was presumably partly a reaction to the start of movement, which in turn,again lead to greater displacements. It appears therefore that a period of time passed before change in one of the parameters directly influenced the other. The rise in the phreatic surface was rapidly followed by a general decline towards mean values. As this dissipation occurred so movement ceased. The continuing fall in the phreatic level following the cessation of movement increased the stability of the slope at its new angle of repose.

The toe slope piezometer showed no evidence of major response to these upslope conditions. There was a slight increase in the head of water which may be due to loading but, as with values recorded during 'multiple' (stick-slip) movements this may well have been due to the general rising water level.

A model can be proposed for movements of this type. Conditions on the slope become increasingly unstable, marked particularly by a rise in the phreatic surface. As porewater pressures rise movement commences with a number of small displacements. These affect the head of water which rises and further increases movement rates. The increased movement subsequently results in porewater pressure dissipation and this finally leads to a cessation of displacements. A complex sequence of interactions therefore occur on the slope.

The example presented above is one of a series of such movements recorded during this study (Table 5.14). These are most common when general ground conditions are particularly favourable for movement, namely at the time of maximum seasonal slope saturation and pore pressure rise.

### 5.7.3 Surge Movements

The third type of displacement identified from these results is termed 'surge' movement. The case study presented here is shown in Figure 5.45 and Figure 5.46. It is an ideal example of the advantages gained in using this type of recording equipment. Data highlights rapid movement of 3.0 m in the centre of the mudslide, which occurred within the space of 20 minutes and with the largest portion of this movement occurring in a five minute period. At four points, small preliminary displacements occurred immediately before the main surge. The main movement was particularly rapid in the centre of the slide. During the main period of advance, all pins showed similar displacements with the major surge starting at all but one point (pin 4) in the space of 10 minutes. Once displacements halted the centre of the mudslide was found to have moved the furthest, with decreasing cumulative movements away from this point. However, changes surprisingly started first close to the lateral shear and at one side of the mudslide (pin 7) much of the slippage has occurred before adjacent material started to move. This phenomenon remains unexplained but a number of hypotheses can be suggested, although none can be demonstrated by the results. Local site conditions, such as fissures and reduced porewater pressures for example, may have occurred at the points where movement was

TABLE 5.14

Details of 'graded movement' pin displacements at Worbarrow Bay

| START OF EVENT | | END OF EVENT | | LOGGED DISPLACEMENT (m) BY POTENTIOMETER | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Date | Time | Date | Time | Pot 1 | Pot 2 | Pot 3 | Pot 4 | Pot 5 | Pot 6 | Pot 7 | Pot 8 |
| 26.10.84 | 1415 | 27.10.84 | 1845 | - | 0.16 | 0.26 | 0.25 | 0.27 | 0.26 | 0.14 | - |
| 30.10.84 | 1530 | 31.10.84 | 1445 | - | 0.14 | 0.27 | 0.19 | 0.25 | 0.26 | 0.11 | - |
| 16.11.84 | 1115 | 17.11.84 | 2000 | - | 0.26 | 0.26 | 0.30 | 0.27 | 0.29 | 0.26 | - |
| 23.11.84 | 1745 | 24.11.84 | 1300 | - | 0.45 | 0.46 | 0.40 | 0.42 | 0.36 | 0.34 | - |
| 12.12.84 | 1545 | 12.12.84 | 2345 | - | 0.18 | 0.31 | 0.10 | 0.14 | 0.21 | 0.38 | - |
| 17.12.84 | 0230 | 18.12.84 | 1215 | - | 0.22 | 0.36 | 0.06 | 0.20 | 0.25 | 0.50 | - |
| 23.12.85 | 1000 | 24.12.84 | 0500 | - | 0.20 | 0.22 | 0.32 | 0.34 | 0.25 | 0.14 | - |

Note: First mudslide surge occurred on 6 January 1985.
Second mudslide surge occurred on 8 January 1985, destroying gantry.

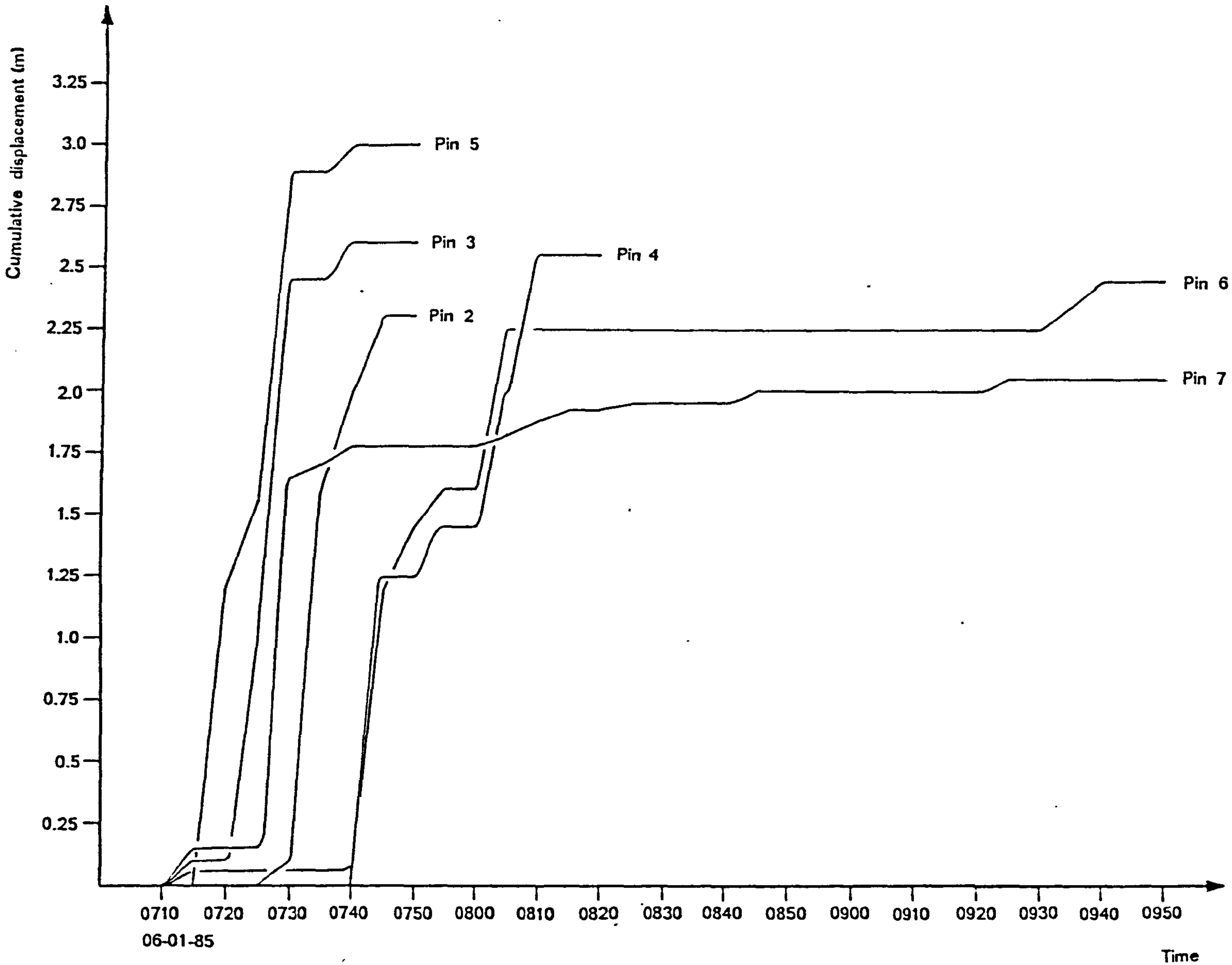



FIGURE 5.45

Example of 'surge' slip pin displacements recorded on the mudslide at Worbarrow Bay

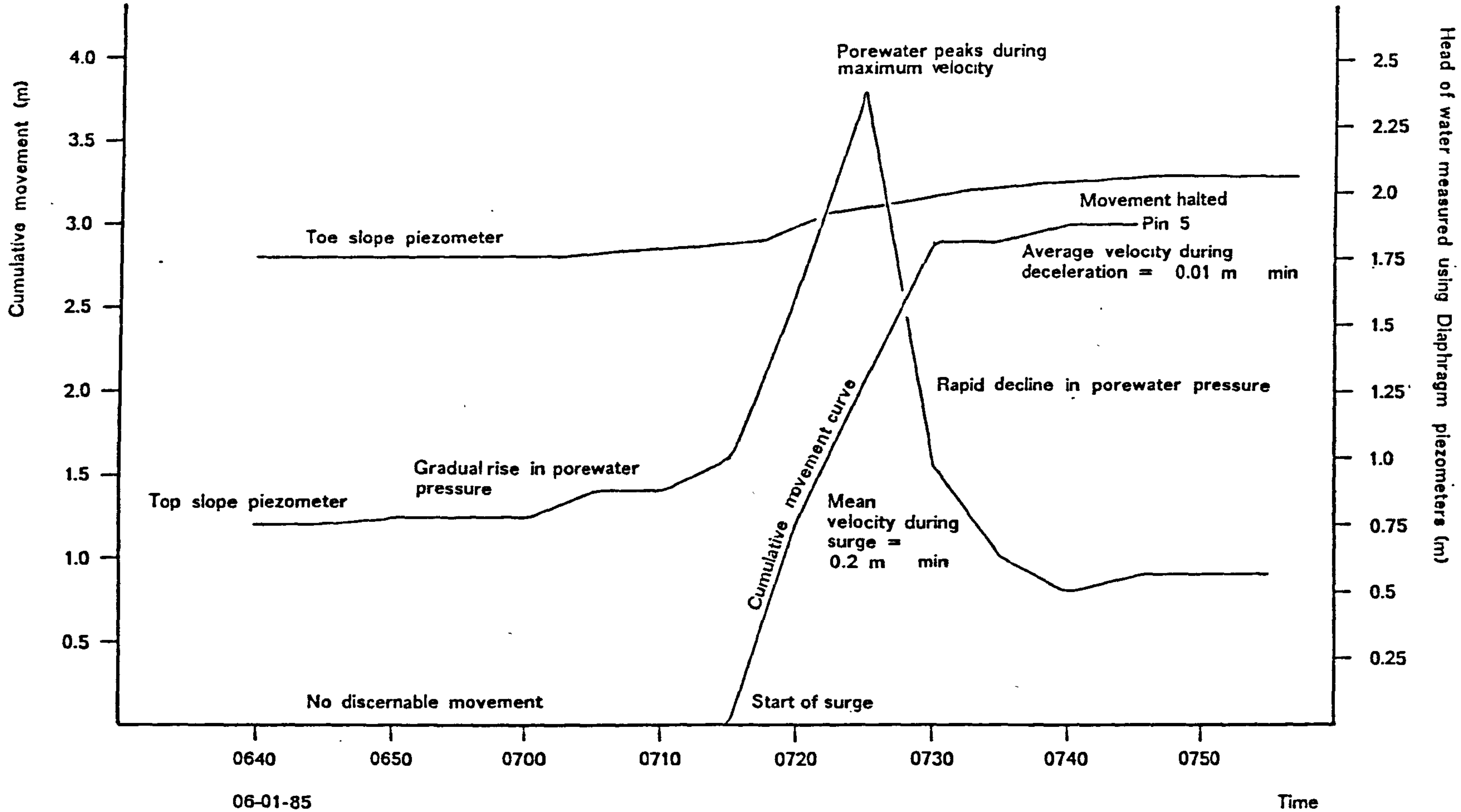



FIGURE 5.46

Relationship between pore water pressure and 'surge' slip displacements of the central pin on the mudslide at Worbarrow Bay initially retarded. Closure of fissures upon initial displacements would have permitted a rapid rise of porewater pressures to the critical values necessary to initiate movement. Indeed the closure of local fissures may explain local diversities in movement. A second hypothesis is that differential zones of movement occur across a mudslide at any one time, and that through time the velocity of each zone changes, so that ultimately total movements conform to standard parabolic curves. In this example, the centre and edges of the mudslide started moving first, while areas between these three points remained relatively stable. This situation reversed during latter movements. Alternatively, this phenomenon may be a site specific characteristic due, for example, to the lithological variations.

Further details emerge from an examination of changes in the phreatic surface. The movement surge was preceded by gradually increasing porewater pressures at both the top and the toe of the slope, although changes were not as clearly marked in the latter instance. Immediately before the surge, the rate of rise of the phreatic surface was faster than it was for 'multiple' (stick-slip) and 'graded' movements. As movement commenced, the phreatic surface was seen to increase dramatically by almost 1.5 m in less than 10 minutes at the top of the slope, subsequently dropping in a similar time period to values approaching those recorded immediately before the movement event. The piezometric level continued to fall over the next 10 - 15 minutes, reaching a constant level lower than that recorded before the surge. At the same time the rate of porewater rise increased at the toe of the slope, this continuing after the cessation of movement. Displacements of this magnitude therefore had a clear effect throughout the mudslide. While the toe slope piezometer did not show the same peaked characteristic of the top slope record, the overall phreatic level was higher. At the toe of the slope therefore, the phreatic curve represented the influence of increased loading from upslope areas of the mudslide.

These results can be used to propose a general model for this type of slope movement. Porewater pressures gradually rise to values which are critical to the overall stability of the mudslide. Variables change sufficiently to cross critical thresholds above which greatly increased movements occur. As a movement surge commences, porewater pressures in the surrounding material rise rapidly to a peak but they then dissipate rapidly leading to a cessation of movement as the mudslide finds its new angle of stability. Porewater pressures finally level out at a new value below that recorded before the slip. At the mudslide foot material remains <u>in-situ</u>, since the parameters controlling movement do not reach critical levels relative to the lower slope angles. Porewater pressures do rise, however, as movement higher up the slope increases the toe slope loading. This loading is bound to decrease the stability of the mudslide at its foot and must create instability at some time to allow the toe lobe to advance across the beach. The conditions for such movements are, however, unknown. The cumulative effect of a number of such events would, for example, be bound to cause large slips lower down the mudslide.

In this model it is therefore suggested that movements in the upper part of the slide generate critical porewater pressures in the middle of the slope. This mobilises a large portion of the track and a consequent rise in porewater pressure occurs on the central area. In the present case study the movements do not load the toe sufficiently to cause displacements at the lower end of the mudslide. Loading can be sufficient, however, to mobilise the whole mass movement. It is therefore suggested that the initial perturbation at the top of the slope is propagated downslope as a positive feedback mechanism. It is pertinent that Iveson (1985) has recently suggested a new model for slow moving landslides which portrays similar characteristics.

Following the work of Hutchinson & Bhandari (1973) it might be expected that the above argument should be reversed and the rapid porewater pressure rise be regarded as the cause of, rather than the response to, movement. In the Hutchinson & Bhandari model, rapid porewater pressure rise is explained by undrained loading at the head of the mudslide from material falling from the feeder slope. This research at Worbarrow Bay shows, however, that the loading occurred in November 1984, some two months before the movements under discussion. Undrained loading is therefore not the cause of the surge. Nevertheless the principle still applies, although in this case it is the upper part of the mudslide which rapidly loads the central section. In other words, there is an inbuilt threshold in the system where movement itself generates more rapid failure. It remains to be seen, however, whether a different timing rear scarp supply would have generated undrained loading. This implies that the mudslide itself must be at or close to critical conditions for undrained loading to be effective. It also suggests that maximum instability would occur if undrained loading from the rear scar coincided with critical conditions of self loading within the track itself.

Two 'surge' movements were recorded during this monitoring phase. One has been presented here. The second occurred two days later and resulted in movement along the whole slope, destroying the gantry supporting the movement sensors. Consequently the results of this second surge are not open to detailed interpretation. What appears to have happened, however, is that the event described here occurred as a preliminary to the following surge. Gradual increases in porewater pressure at the toe of the slope, enhanced by a sudden phreatic rise caused by loading on 6th January 1985, lead to critical conditions being exceeded,causing slip along the whole of the mudslide.

### 5.7.4 Other Movements

Much of the data logger record showed no change. Occasionally variations occurred in the results of one logger step. These were apparently random events through time and space, probably representing slow internal distortions to the mass, plasticity effects and the results of gravitational forces. Such small movements are bound to show up from time to time on the overall record. Examples of these occurring over the period of a fortnight are presented in Table 5.15.

### 5.7.5 Discussion

The instrumentation and monitoring used to obtain a detailed record of mudslide movements gives a picture of the total change dividing

TABLE 5.15

Random small scale movements occurring over one fortnight period at Worbarrow Bay

| DATE | TIME | POTENTIOMETER RECORDING MOVEMENT AND LOGGED DISPLACEMENT (m) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | POT 1 | POT 2 | POT 3 | POT 4 | POT 5 | POT 6 | POT 7 | POT 8 |
| 18-10-84 | 0000 | | | | | | | | |
| 19-10-84 | 1750 | | | | | 1.0 | | | |
| 19-10-84 | 1920 | | 1.0 | | | | | | |
| 21-10-84 | 2145 | | | | | | | 1.0 | |
| 22-10-84 | 0510 | | | | | | | 1.0 | |
| 22-10-84 | 0750 | | | 1.0 | | | | | |
| 24-10-84 | 0640 | | | | | 1.0 | | | |
| 25-10-84 | 0025 | | | | | 1.0 | | | |
| 26-10-84 | 1140 | | 1.0 | | | | | | |
| 28-10-84 | 0755 | | | | | | 1.0 | | |
| CUMULATIVE DISPLACEMENT (cm) | | - | 2.0 | 1.0 | - | 3.0 | 1.0 | 2.0 | - |

into a number of different components. Clearer definition of these types of movement has been possible, highlighting their specific characteristics, when they occur and how important they are to overall cumulative seasonal trends. Much of this detail has not been recognised in previous studies. This highlights the useful application of modern data collection techniques, the adoption of a detailed temporal sampling framework and the careful identification from the resulting data of significant changes to the record.

Such results can be seen to complement records kept over longer periods,collected using a less regular sampling framework. A study of this nature requires both levels of investigation, long term changes highlighting general trends and regular records providing the details necessary for a full and comprehensive analysis of the different types of movement within overall seasonal changes.

## 5.8 CONCLUSION

The results presented here are a comprehensive statement of the slope movements recorded along the coastal cliffs in the Wealden Beds. Clear spatial and temporal patterns can be identified. Not only do the results provide important information relative to this particular study area but also general conclusions relating to the morphology and processes of mudslides have been identified. In part, these follow the trends identified in previous studies (Hutchinson et al., 1974, Craig, 1979) but much new detail has also been presented,which has clarified the exact nature of mudslide movements as seasonal and short-term conditions change.

# CHAPTER VI STABILITY ANALYSIS

## 6.1 INTRODUCTION

This thesis has so far centred on cliff morphology and coastline recession, mechanisms of failure, the parameters governing slope stability and slope movements. Although directly relevant to the aims of this study, the results of these investigations are only partially useful in determining the likelihood or potential of a slope to failure. Stability analysis is a long established method of providing a quantitative statement on the stability of a slope by considering its geometry and mechanical properties (Graham, 1984). Although such methods have their foundation in engineering soil mechanics (for example, Terzaghi & Peck, 1967; Lambe & Whitman, 1979; Chandler et al., 1976) they have been adopted by geomorphologists as a quantitative technique, permitting the detailed assessment of land-forms by considering the geometry of a slope in conjunction with measured field conditions and geotechnical properties (for example, Campbell, 1975; Rogers & Selby, 1980; Selby 1967).

Slope stability analysis is particularly relevant to this research. A quantitative statement can be made on the potential stability of the coastal cliffs using field and laboratory data available from the geomorphological and geotechnical investigations. The results can be used as a predictive technique while the previously discussed analysis does not permit this. Stability analysis also provides a unifying link for other parts of this study including geomorphological, geolog-ical and geotechnical investigations. Finally, due to the nature of this technique, general site conditions are considered and consequently many potential sources of error such as variable lithology are minimised. Despite these advantages, however, a number of details must be borne in mind. All stability analyses can be restrictive in their application and are not a tool to analyse rates of movement. This has been previously studied, however (chapter V) and in this light stability analysis further complements this study. Stability analysis is not the principal aim of this study and it must therefore be regarded as complementary to, rather than the objective of, the overall investigation. The specific techniques, as used in this thesis, involve a 2-dimensional analysis of 3-dimensional problems, using variables which can differ considerably and be difficult to measure. These points emphasise the care required when interpreting the results and the major advantages of a detailed mapping programme, which can be used to identify other critical parameters not included in a standard geotechnical investigation. Although there are many slope stability models, seldom more than one is of relevance to any specific study. An important pre-requisite to the use of this type of analysis is, therefore, the careful identification of the most suitable model.

## 6.2 CHOICE OF MODEL

Due to the contrasting lithologies examined in this study two separate stability analyses are required. No single model is sufficiently well developed to consider materials as different as those studied here, since the major controls on stability change between engineering soils such as the Wealden Beds and rocks like the Portland Limestone and Chalk.

### 6.2.1 The Wealden Beds

The method developed by Janbu (1973) was chosen for the analysis of the Wealden Beds because all the required parameters are measurable at each of the field sites. Slope geometries and material geotechnical properties have all been successfully quantified for example, and the Janbu (1973) technique does not require the slip surface to be either flat or of regular geometry. As will be shown, at some of the sites assumptions regarding the location of the slip surface between measured points were not always correct and some adjustment of their shape was required. Using this technique, all input variables can be held constant or an individual parameter can be changed to study its effect on overall slope stability. Pore water pressure is an ideal example.

A computer program was used for the analysis. This had a number of advantages. Once the basic parameters which can be accurately quantified have been entered, other data of a more variable nature, such as the position of the phreatic surface, can be varied. Extrapolation of the shear surface was required between points of known depth. This analysis is capable of handling such inferences and the shear surface was in fact modified, using the program, to that shape thought to be most representative of overall site conditions. By using a careful combination of measured parameters and predicted data, an attempt can be made to evaluate the current state of stability of the monitored coastal slopes. For example, the level of the piezometric surface seemed to be of major importance in the initiation of phases of movement, while factors such as unloading following the removal of the slope material appeared to be less significant.

Despite the advantages of the Janbu method a number of points should be noted. Graham (1984) specifically notes that cliff regression is an area where uncertainty still exists in attempting to predict failure and that additional care must be taken in applying stability analysis to such situations. All stability models are simplifications of reality, which exclude some parameters which may be important and indeed Selby (1982), for example, suggests an error factor of $\pm$ 10% and that such variability should be supplemented by mapping, which can enhance the knowledge of local conditions. The details presented in Chapter III serve this purpose. Finally, many parameters have more than one identifiable value. Material strength is one example (Skempton, 1964). In these cases care has to be taken to select the measurement most appropriate to the field conditions.

The specific program used here is utilised commercially by engineering geologists (Cobb, pers. comm.). Consequently, it is not possible to list details of the analysis routines. Data input and format requirements (section 6.3.1) can be presented however. The results are essentially the same as any calculated using the Janbu (1973) technique, with the added advantage that since a large number of analyses can be conducted over a short space of time, a variety of Factors of Safety can be presented for different field conditions.

### 6.2.2 Portland Limestone

Stability analysis for slopes formed on hard rock are frequently less reliable than for engineering soils because geological conditions at field sites can be complex. While detailed investigation techniques are well developed, the application of engineering procedures to geomorphological problems is not as widely practised in rock mechanics as it is in soil mechanics. A number of methods of investigating rockfalls currently exist (for example, Arrowsmith, 1971; Fookes & Sweeney, 1976; Kojan & Hutchinson, 1978; Peckover & Kerr, 1977). A two-stage analysis was chosen for hard rock materials, utilising stereographic projection and a computer based analysis designed to investigate the stability of free faces.

Stereographic projection forms a natural link between the discontinuity analysis discussed in chapter IV and more detailed computer-based slope stability calculations. In addition, discontinuities in rock masses have long been recognised as the major control on stability (Whalley, 1984). Thus a detailed assessment of this specific parameter is required before other controlling variables can be considered. There are two aspects to the stability of rock slopes; the actual mode of failure and the likelihood of collapse irrespective of the mechanism by which it occurs. The two stage approach adopted here tackles both these problems. The principles and application of stereographic projection are widely reported (Hobbs et al., 1976; Hoek, 1973; Phillips, 1979; Priest, 1985; Brown, 1981; Hoek & Bray, 1981) and require no further discussion here. Detailed slope stability models based on rock mechanics are not as well developed or as numerous as those used in soil mechanics. The model finally chosen is not widely available and was designed to assess the stability of free faces in open-casting mining (Cobb, pers. comm.). Much difficulty was experienced in identifying the most suitable method of analysis. Commonly available techniques were deemed to be less suitable overall, when compared with the chosen analysis method, because they could not incorporate the large number of parameters identified by the joint survey and stereographic analysis programmes, which appear to contribute to cliff stability. It was also necessary for the computer model used to be suitable for the analysis of tetrahedral blocks. The chosen model considers the free face and as such can be applied to all situations, both natural and man made. The combined height and angle of some of the cliffs being studied, particularly where they plunge directly into the sea, made data collection difficult. It was consequently impossible to obtain the details required by some of the alternatively available models. This is not so for the chosen method of stability analysis, where all necessary data can be obtained. Finally, many models include parameters which do not appear to be applicable to this study, primarily because they remain constant between sites. For example, details of cleft water pressure is frequently required, but there is no evidence of this being important here. All parameters which are required in the employed analysis vary considerably along the coast, appear from preliminary analysis to contribute to the cliff stability and are therefore important input requirements.

It was decided that these techniques should be applied to the Portland Limestone only. Mapping, field investigations and the geotechnical study suggest that the Isle of Purbeck Chalk is a highly variable and complex material. It was found to be impossible to accurately assess the stability of the cliffs in the area by using any model which makes a number of assumptions and requires simplification of field conditions. Indeed, it would be naive to think that the use of such a technique on the Chalk would enhance an understanding of slope development and cliff stability. It is more likely that the contrary would be true, with the exclusion of many important parameters which can not be successfully quantified, resulting in a misrepresentation of actual conditions. With these points in mind, it seems inadvisable to present any further interpretation in addition to that resulting from the geotechnical investigation and rock classification (chapter IV). The most suitable conclusions for the Chalk are to be drawn from this broad consideration of material characteristics, rather than a detailed study based on Factors of Safety, when the data required for the calculation of this parameter cannot be adequately determined. In conclusion, while it is clear that the application of both stereographic projection and slope rock face analysis will contribute to an understanding of the Portland Limestone, the same cannot be said about the Chalk.

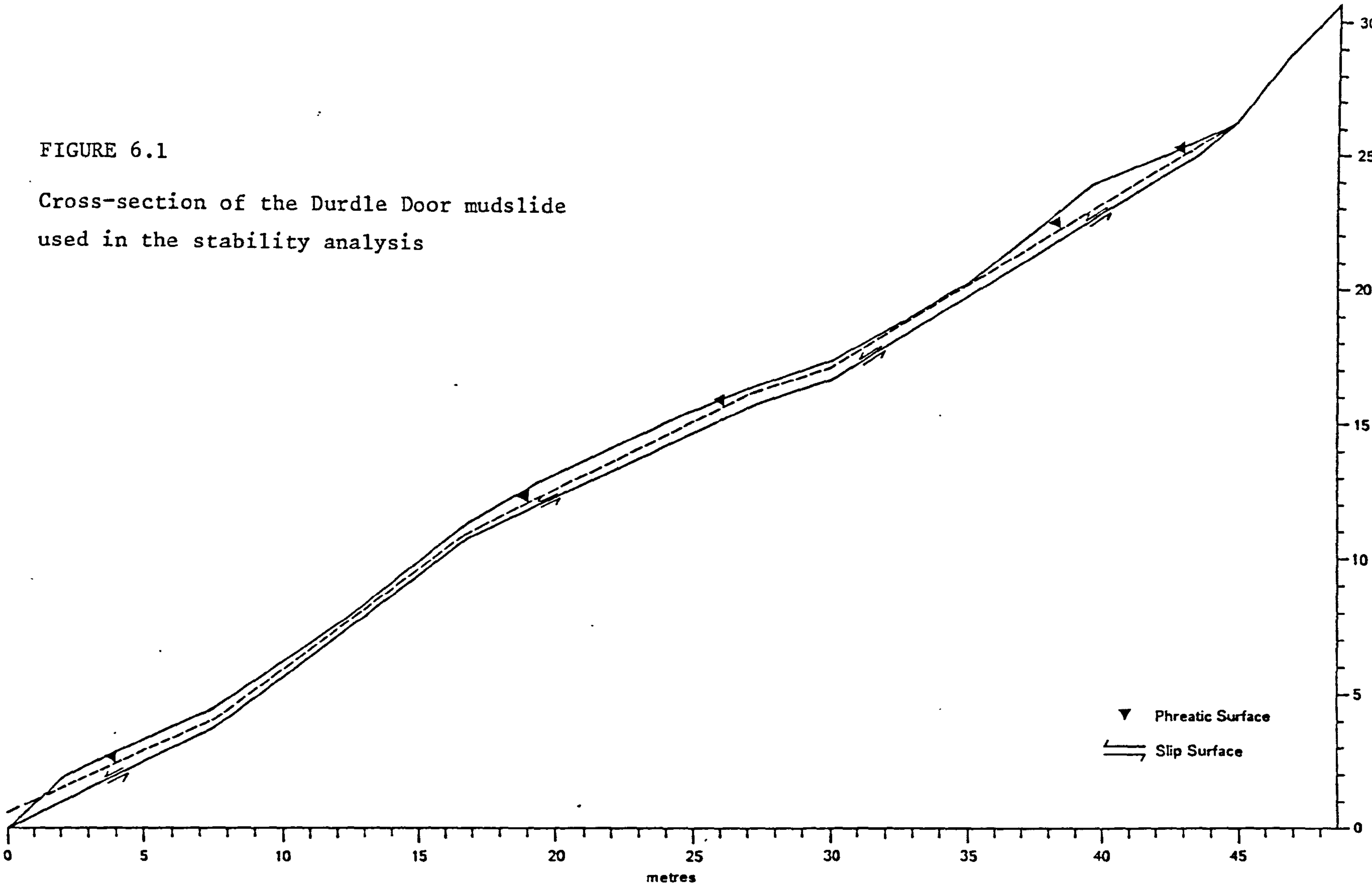
## 6.3 STABILITY ANALYSIS OF THE WEALDEN BEDS

### 6.3.1 Data Requirements and Input Format

Much of the data required for the stability analysis has been previously collected and discussed (chapter IV and V). These include the slope long surface profile, the depth of the slip surface over which the material is currently moving, the level of the phreatic surface particularly at the point in time immediately before the commencement of movement and the material bulk density. The slope geometry requires conversion to a sequence of X-Y co-ordinate points, such that 0,0 represents the leading edge of the cross-section under investigation. All other points along the ground surface, the slip surface and phreatic surface are defined relative to this. The geometrical details were prepared from a series of cross-sections (Figures 6.1-6.4). The actual co-ordinates are shown in the details of the analysis. It is necessary to make a number of preliminary assumptions. The slip

FIGURE 6.1

Cross-section of the Durdle Door mudslide used in the stability analysis



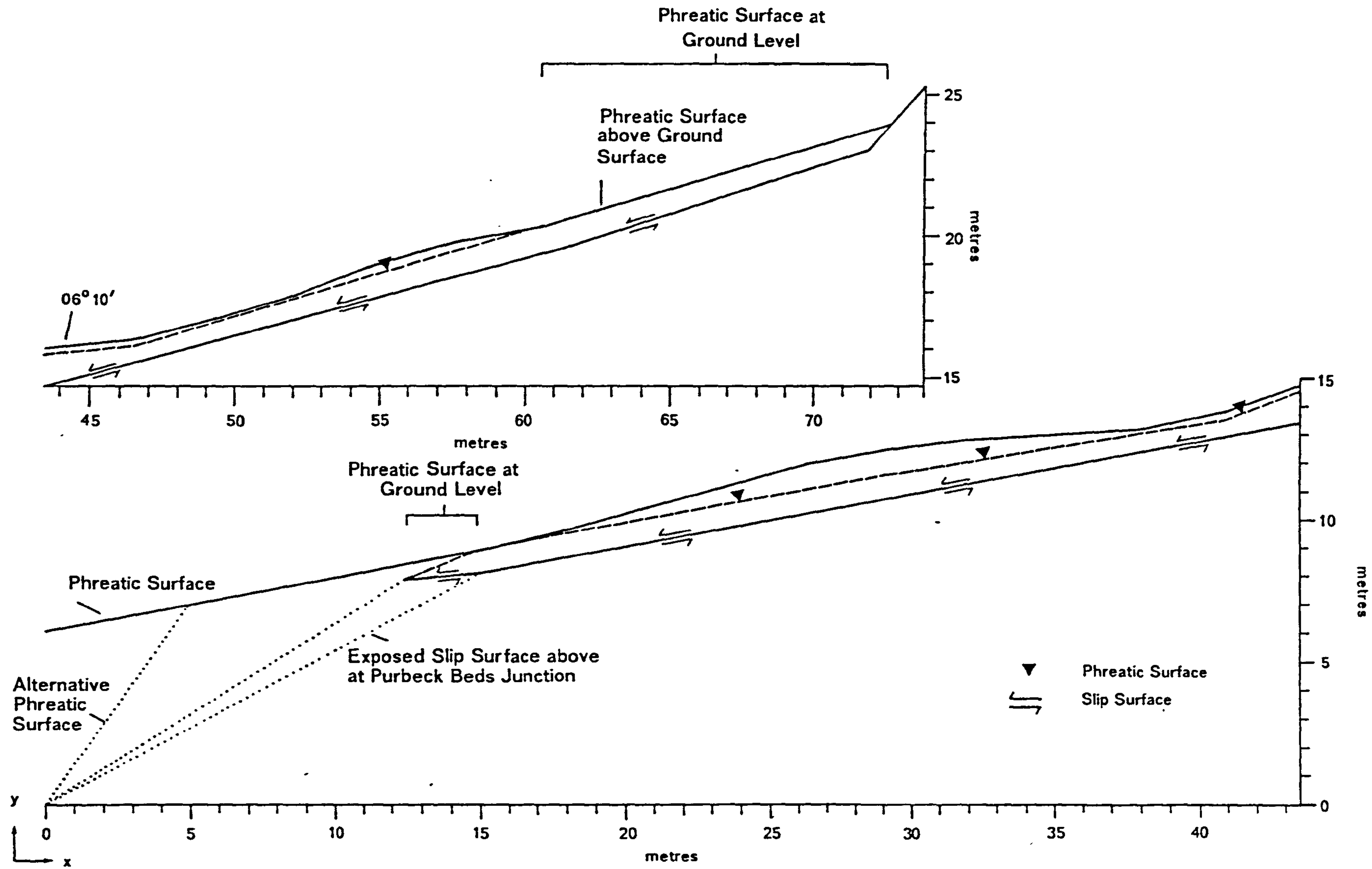
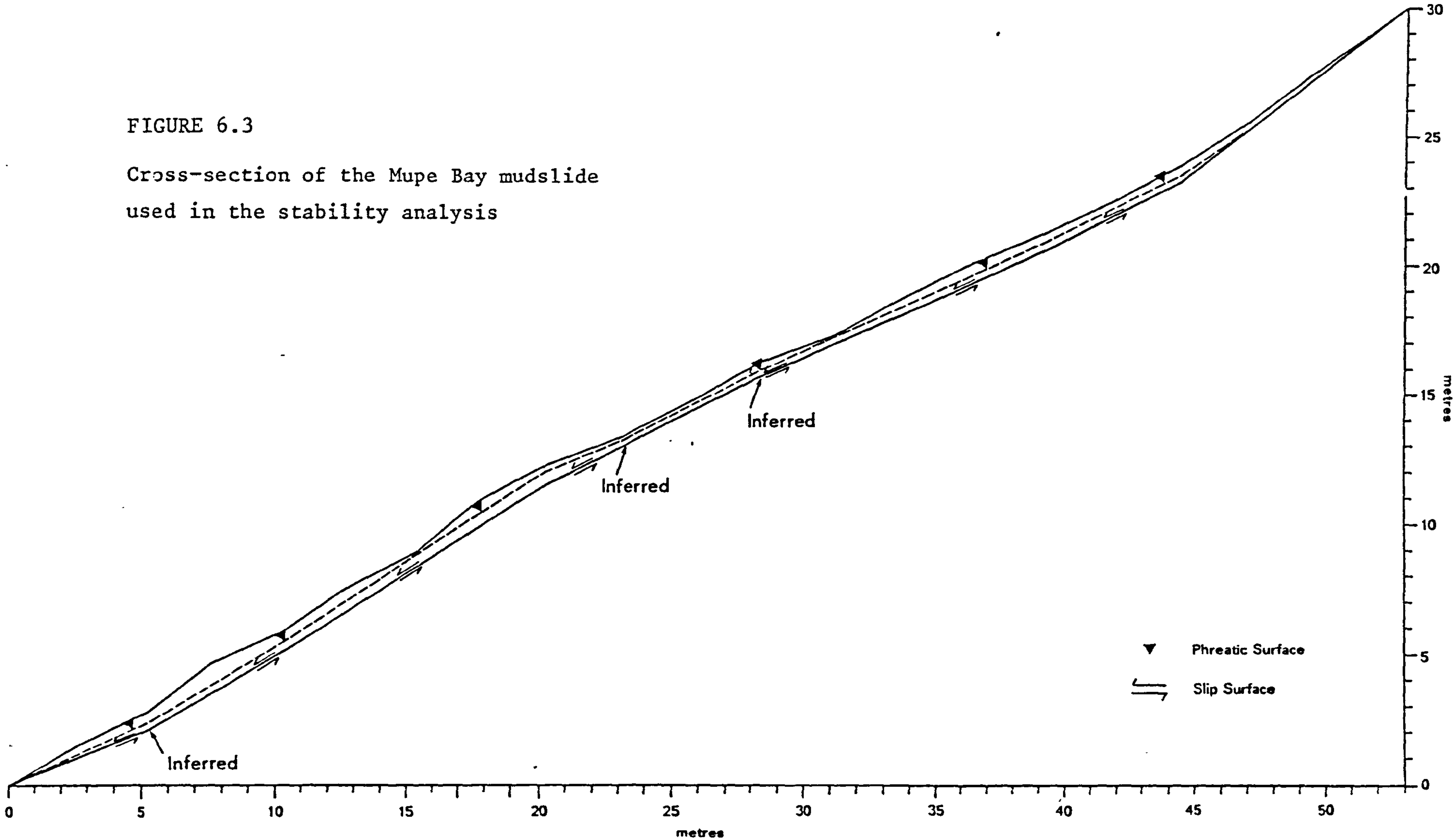



FIGURE 6.2 Cross-section of the Stair Hole mudslide used in the stability analysis

FIGURE 6.3

Cross-section of the Mupe Bay mudslide used in the stability analysis



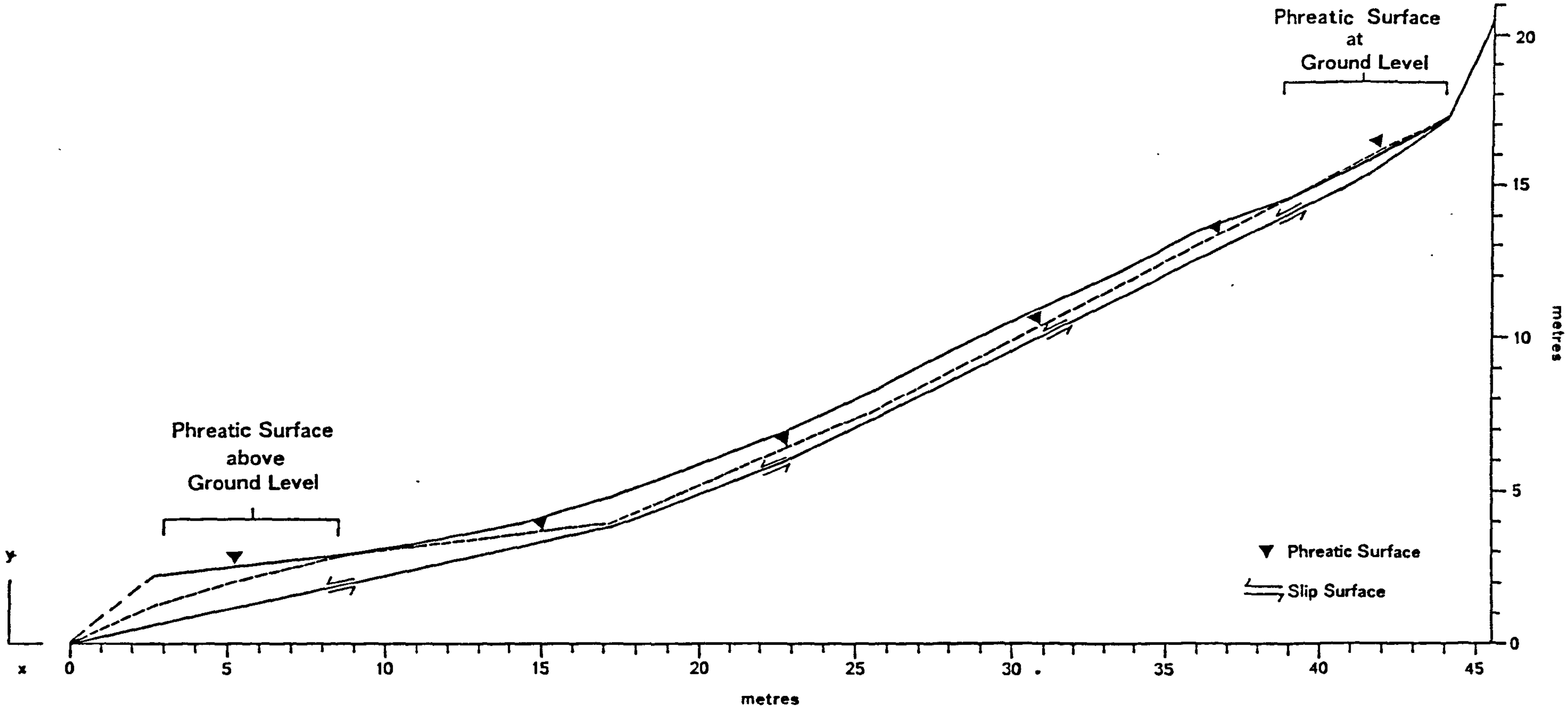



FIGURE 6.4

Cross-section of the Worbarrow Bay mudslide used in the stability analysis surface is assumed to meet the ground surface at the toe of the slide. In some instances this may not be the case but such an error is likely to have little or no influence on the overall result. Where standpipe piezometers are completely full of water, artesian porewater pressures are assumed. By comparing the level of the phreatic surface between piezometers, in conditions when all levels are below the top of the standpipe, estimates can be made of the artesian phreatic surface by relative deduction. Finally, it is assumed that the slip surface runs directly between points of known depth. In some circumstances these details will require modification, but data preparation dictates that a bench mark has to be established from which changes can be made which are likely to be more representative of site conditions.

Stability analysis was conducted at points along the coast on the mudslides previously identified as being most representative of overall site conditions. Although site characteristics have been previously discussed (chapter III) there are details specifically relevant to this stability analysis. The profile at Durdle Door (Figure 6.1, Table 6.1) shows that towards the toe of the slide the slope angle is considerably reduced. The depth of the slip surface therefore had to be inferred. Failure to do this would have resulted in slip surface co-ordinates exceeding those of ground surface. The geomorphological mapping suggests that stratigraphical variations in the lithology control the location of the analysed slide, due to the position of particularly sandy units. The analysis was conducted by altering the values of variables, such as the slip surface depth, with this in mind. In addition the ground surface, slip surface and phreatic surface are all thought to meet at co-ordinate point 0,0. Seaward from this point material moves out over the beach. Although

KEY FOR USE WITH TABLES 6.1 - 6.4

1. Ground surface co-ordinates represent X-Y listings of survey data, from slope toe to slope crest. Refer to Figures 6.1-6.4 for details of slope profiles.

2. Material strength parameters included in this analysis are

i) Material effective cohesion (C)
ii) Angle of shearing resistance (PHI)
iii) Material bulk density

3. The phreatic surface is presented as a series of X-Y co-ordinates representing details of the pore water pressure used in each analysis.

4. The slip or failure surface is given as a series of X-Y co-ordinates from slope toe to slope crest, listing results of the sub-surface investigation program.

5. Slice parameters are chosen in each instance by the program and are based on changes in the slope at the ground surface.

6. P LAYER 1: the horizontal matrix component which in this instance is a range of cohesion values.

7. C LAYER 1: the vertical matrix component which in this instance is a range of angle of internal friction (or angle of shearing resistance) values.

8. The Factors of Safety are presented as matrices, one result being given for each combination of a specific cohesion and angle of internal friction value defined by C, LAYER 1 and P, LAYER 1 respectively

TABLE 6.1a

Durdle Door Stability Analysis - parameter co-ordinates

Ground Surface (Layer Beneath in Brackets)

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | 0.00 | (1) | 2: | 2.60 | 1.50 | (1) | 3: | 5.30 | 2.80 | (1) |
| 4: | 7.65 | 4.65 | (1) | 5: | 10.35 | 5.90 | (1) | 6: | 12.80 | 7.60 | (1) |
| 7: | 15.45 | 8.95 | (1) | 8: | 17.70 | 10.85 | (1) | 9: | 20.35 | 12.30 | (1) |
| 10: | 23.15 | 13.35 | (1) | 11: | 25.80 | 14.70 | (1) | 12: | 28.35 | 16.20 | (1) |
| 13: | 31.20 | 17.20 | (1) | 14: | 33.65 | 18.65 | (1) | 15: | 36.45 | 20.05 | (1) |
| 16: | 39.20 | 21.15 | (1) | 17: | 41.90 | 22.40 | (1) | 18: | 44.50 | 23.85 | (1) |
| 19: | 47.00 | 25.50 | (1) | | | | | | | | |

Strength Parameters

| | PRIMARY | | | SECONDARY | | | |
|---|---|---|---|---|---|---|---|
| | COHESION (kPa) | PHI | DENSITY ($Mgm^{-3}$) | COHESION (kPa) | PHI | IN RANGE LOWER | IN RANGE UPPER |
| LAYER 1: | 0.00 | 14.00 | 1.84 | --- | --- | --- | --- |

Failure Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | 0.00 | 2: | 5.30 | 2.10 | 3: | 10.35 | 5.20 | 4: | 20.35 | 11.60 |
| 5: | 28.35 | 15.65 | 6: | 39.20 | 20.55 | 7: | 44.50 | 23.20 | 8: | 47.00 | 25.50 |

TABLE 6.1b

Durdle Door Stability Analysis - phreatic surface as measured

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | -.10 | 2: | 5.30 | 2.30 | 3: | 10.35 | 2.45 | 4: | 20.35 | 12.00 |
| 5: | 23.15 | 13.10 | 6: | 28.35 | 15.80 | 7: | 39.20 | 20.75 | 8: | 44.50 | 23.40 |
| 9: | 47.00 | 25.40 | | | | | | | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 5.88 | .65 | .92 | 2.50 | 0.00-2.25 | 40.00-22.00 |
| 2 | 10.17 | 1.96 | .50 | 5.30 | 0.00-2.25 | 40.00-22.00 |
| 3 | 9.93 | 1.72 | .45 | 10.85 | 0.00-2.25 | 40.00-22.00 |
| 4 | 7.85 | 1.57 | .51 | 8.00 | 0.00-2.25 | 40.00-22.00 |
| 5 | 13.41 | .25 | .64 | 10.00 | 0.00-2.25 | 40.00-22.00 |
| 6 | 16.34 | .07 | .61 | 5.05 | 0.00-2.25 | 40.00-22.00 |
| 7 | 7.47 | .65 | .40 | 5.30 | 0.00-2.25 | 40.00-22.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 9.49 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | 1.15 | 1.22 | 1.29 | 1.36 | 1.43 | 1.50 | 1.59 | 1.68 | 1.77 | 1.86 |
| 2.00 | 1.10 | 1.16 | 1.23 | 1.30 | 1.37 | 1.45 | 1.53 | 1.62 | 1.71 | 1.80 |
| 1.75 | 1.04 | 1.11 | 1.17 | 1.24 | 1.32 | 1.39 | 1.47 | 1.55 | 1.65 | 1.74 |
| 1.50 | .98 | 1.05 | 1.12 | 1.19 | 1.26 | 1.33 | 1.42 | 1.51 | 1.59 | 1.69 |
| 1.25 | .93 | .99 | 1.06 | 1.13 | 1.20 | 1.28 | 1.36 | 1.44 | 1.54 | 1.63 |
| 1.00 | .87 | .93 | 1.00 | 1.07 | 1.15 | 1.22 | 1.30 | 1.38 | 1.47 | 1.56 |
| .75 | .81 | .88 | .95 | 1.02 | 1.09 | 1.17 | 1.25 | 1.33 | 1.42 | 1.52 |
| .50 | .76 | .82 | .89 | .96 | 1.03 | 1.11 | 1.19 | 1.27 | 1.36 | 1.45 |
| .25 | .70 | .77 | .84 | .91 | .98 | 1.06 | 1.14 | 1.22 | 1.30 | 1.40 |
| 0.00 | .65 | .71 | .78 | .85 | .92 | 1.00 | 1.08 | 1.16 | 1.25 | 1.34 |
| | 22.00 | 24.00 | 26.00 | 28.00 | 30.00 | 32.00 | 34.00 | 36.00 | 38.00 | 40.00 P LAYER 1 |

TABLE 6.1c

Durdle Door Stability Analysis - phreatic surface 0.1 m higher

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 5.88 | 1.47 | .92 | 2.50 | 0.00-2.25 | 40.00-22.00 |
| 2 | 10.17 | 2.94 | .50 | 5.30 | 0.00-2.25 | 40.00-22.00 |
| 3 | 9.93 | 2.70 | .45 | 10.85 | 0.00-2.25 | 40.00-22.00 |
| 4 | 7.85 | 2.55 | .51 | 8.00 | 0.00-2.25 | 40.00-22.00 |
| 5 | 13.41 | 4.17 | .64 | 10.00 | 0.00-2.25 | 40.00-22.00 |
| 6 | 16.34 | 3.19 | .61 | 5.05 | 0.00-2.25 | 40.00-22.00 |
| 7 | 7.47 | 1.47 | .40 | 5.30 | 0.00-2.25 | 40.00-22.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 7.68 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | .98 | 1.04 | 1.09 | 1.15 | 1.20 | 1.26 | 1.32 | 1.38 | 1.44 | 1.51 |
| 2.00 | .93 | .99 | 1.04 | 1.09 | 1.14 | 1.20 | 1.26 | 1.32 | 1.39 | 1.46 |
| 1.75 | .87 | .92 | .97 | 1.03 | 1.09 | 1.14 | 1.19 | 1.26 | 1.33 | 1.40 |
| 1.50 | .81 | .86 | .92 | .98 | 1.02 | 1.08 | 1.14 | 1.20 | 1.26 | 1.34 |
| 1.25 | .76 | .81 | .86 | .91 | .97 | 1.02 | 1.08 | 1.14 | 1.21 | 1.28 |
| 1.00 | .70 | .75 | .80 | .86 | .91 | .97 | 1.03 | 1.09 | 1.15 | 1.22 |
| .75 | .65 | .70 | .75 | .80 | .85 | .91 | .97 | 1.03 | 1.09 | 1.16 |
| .50 | .59 | .64 | .69 | .75 | .80 | .85 | .91 | .97 | 1.04 | 1.11 |
| .25 | .54 | .59 | .63 | .68 | .74 | .80 | .86 | .92 | .98 | 1.05 |
| 0.00 | .48 | .53 | .58 | .63 | .69 | .74 | .80 | .86 | .93 | 1.00 |
| | 22.00 | 24.00 | 26.00 | 28.00 | 30.00 | 32.00 | 34.00 | 36.00 | 38.00 | 40.00 P LAYER 1 |

TABLE 6.1d

Durdle Door Stability Analysis - phreatic surface 0.2 m higher

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | .10 | 2: | 5.30 | 2.50 | 3: | 10.35 | 5.65 | 4: | 20.35 | 12.20 |
| 5: | 23.15 | 13.30 | 6: | 28.35 | 16.00 | 7: | 39.20 | 20.95 | 8: | 44.50 | 23.45 |
| 9: | 47.00 | 25.60 | | | | | | | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.05 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | SHEAR STRENGTH PHI |
|---|---|---|---|---|---|---|
| 1 | 5.88 | 1.72 | .92 | 2.50 | 0.00-2.25 | 40.00-22.00 |
| 2 | 10.17 | 3.19 | .50 | 5.30 | 0.00-2.25 | 40.00-22.00 |
| 3 | 9.93 | 3.68 | .45 | 10.85 | 0.00-2.25 | 40.00-22.00 |
| 4 | 7.85 | 3.53 | .51 | 8.00 | 0.00-2.25 | 40.00-22.00 |
| 5 | 13.41 | 5.15 | .64 | 10.00 | 0.00-2.25 | 40.00-22.00 |
| 6 | 16.34 | 4.17 | .61 | 5.05 | 0.00-2.25 | 40.00-22.00 |
| 7 | 7.47 | 2.45 | .40 | 5.30 | 0.00-2.25 | 40.00-22.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 6.82 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | .91 | .95 | .99 | 1.03 | 1.09 | 1.14 | 1.18 | 1.23 | 1.28 | 1.34 |
| 2.00 | .85 | .89 | .93 | .99 | 1.02 | 1.07 | 1.12 | 1.17 | 1.23 | 1.29 |
| 1.75 | .79 | .83 | .88 | .92 | .97 | 1.01 | 1.06 | 1.12 | 1.17 | 1.23 |
| 1.50 | .74 | .78 | .82 | .87 | .91 | .96 | 1.01 | 1.06 | 1.11 | 1.17 |
| 1.25 | .68 | .72 | .77 | .81 | .86 | .90 | .95 | 1.00 | 1.06 | 1.11 |
| 1.00 | .63 | .67 | .71 | .76 | .80 | .85 | .90 | .95 | 1.00 | 1.06 |
| .75 | .57 | .61 | .66 | .70 | .74 | .79 | .84 | .89 | .95 | 1.00 |
| .50 | .52 | .56 | .60 | .64 | .69 | .73 | .78 | .84 | .89 | .95 |
| .25 | .46 | .50 | .54 | .59 | .63 | .68 | .73 | .78 | .83 | .89 |
| 0.00 | .40 | .44 | .49 | .53 | .58 | .62 | .67 | .72 | .78 | .83 |
| | 22.00 | 24.00 | 26.00 | 28.00 | 30.00 | 32.00 | 34.00 | 36.00 | 38.00 | 40.00 P LAYER 1 |

this beach lobe could have been included in the analysis (assuming zero pore water pressure), the results are more likely to be representative of site conditions if this is ignored, due to its small extent relative to the overall slide.

At Stair Hole (Figure 6.2, Table 6.2) variations in the ground surface profile do not mirror the geometry of the slip surface. The former undulates considerably, while the latter has developed as a smooth plane. Consequently, the proximity of slip surface and ground surface varies. Associated with this also are differences in the phreatic surface. Despite the ground surface profile comprising 27 sections, there appear to be 3 main sub-divisions, a steep toe slope zone, a gently inclined main track and a flatter rear slope section. The steep toe slope zone represents sliding of the Wealden Beds over the Purbeck. This 18 m section was not included in the analysis for three reasons. First, during movement, heavily mixed, argillaceous debris slides rapidly over this surface and often breaks away from the true toe of the mudslide, which is therefore considered to end at the top of this slope section. Secondly, including this section of the slope would have considerable effect on the overall stability analysis likely to be unrepresentative of mean site conditions and thirdly, it is impossible to measure many of the required properties across this surface since it remains bare for much of the year and merely acts as a plane over which the mudslide debris descends to the beach. Water levels overtop the top and toe slope piezometers during the wettest seasons, suggesting artesian pore water pressures. A variety of depths for the phreatic surface were used to investigate this. The ground, slip and phreatic surfaces were all assumed to meet at the head of the slide, while at the toe, the ground and slip surfaces were recorded by

TABLE 6.2a

Stair Hole Stability Analysis - parameter co-ordinates

Ground Surface (Layer Beneath in Brackets)

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 7.85 | (1) | 2: | 15.00 | 8.95 | (1) | 3: | 17.85 | 9.60 | (1) |
| 4: | 20.65 | 10.40 | (1) | 5: | 23.45 | 11.10 | (1) | 6: | 26.30 | 11.95 | (1) |
| 7: | 29.15 | 12.45 | (1) | 8: | 32.05 | 12.85 | (1) | 9: | 35.00 | 13.05 | (1) |
| 10: | 37.90 | 13.20 | (1) | 11: | 40.80 | 13.75 | (1) | 12: | 43.55 | 14.70 | (1) |
| 13: | 46.45 | 15.00 | (1) | 14: | 49.30 | 15.70 | (1) | 15: | 52.10 | 16.55 | (1) |
| 16: | 54.85 | 17.70 | (1) | 17: | 57.65 | 18.45 | (1) | 18: | 60.55 | 18.95 | (1) |
| 19: | 63.40 | 19.50 | (1) | 20: | 66.30 | 20.00 | (1) | 21: | 69.10 | 20.90 | (1) |
| 22: | 71.95 | 21.70 | (1) | 23: | 73.85 | 23.90 | (1) | | | | |

Strength Parameters

| | PRIMARY | | | SECONDARY | | | |
|---|---|---|---|---|---|---|---|
| | COHESION (kPa) | PHI | DENSITY ($Mgm^{-3}$) | COHESION (kPa) | PHI | IN RANGE LOWER | UPPER |
| LAYER 1: | 0.00 | 10.00 | 1.53 | --- | --- | --- | --- |

Failure Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 7.85 | 2: | 15.00 | 8.10 | 3: | 29.15 | 10.65 | 4: | 43.55 | 13.45 |
| 5: | 60.55 | 18.00 | 6: | 71.95 | 21.70 | | | | | | |

TABLE 6.2b

Stair Hole Stability Analysis - phreatic surface as measured

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 8.45 | 2: | 15.00 | 8.95 | 3: | 29.15 | 11.55 | 4: | 40.80 | 13.45 |
| 5: | 43.55 | 14.60 | 6: | 46.45 | 14.80 | 7: | 60.55 | 18.75 | 8: | 63.40 | 19.50 |
| 9: | 66.30 | 20.00 | 10: | 69.10 | 20.90 | 11: | 71.95 | 21.70 | 12: | 73.85 | 23.90 |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 4.94 | 2.97 | .32 | 11.40 | 0.00-2.25 | 26.00-8.00 |
| 2 | 14.66 | 6.83 | .27 | 17.00 | 0.00-2.25 | 26.00-8.00 |
| 3 | 18.41 | 7.27 | .19 | 14.40 | 0.00-2.25 | 26.00-8.00 |
| 4 | 20.73 | 8.58 | .18 | 14.15 | 0.00-2.25 | 26.00-8.00 |
| 5 | 6.39 | 7.11 | .09 | 2.70 | 0.00-2.25 | 26.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 8.15 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | 1.09 | 1.17 | 1.26 | 1.35 | 1.44 | 1.54 | 1.63 | 1.73 | 1.83 | 1.94 |
| 2.00 | 1.00 | 1.09 | 1.18 | 1.27 | 1.36 | 1.45 | 1.55 | 1.65 | 1.75 | 1.85 |
| 1.75 | .92 | 1.01 | 1.10 | 1.19 | 1.28 | 1.37 | 1.47 | 1.57 | 1.67 | 1.77 |
| 1.50 | .84 | .93 | 1.01 | 1.10 | 1.20 | 1.29 | 1.39 | 1.48 | 1.58 | 1.69 |
| 1.25 | .76 | .84 | .93 | 1.02 | 1.11 | 1.21 | 1.30 | 1.40 | 1.50 | 1.61 |
| 1.00 | .67 | .76 | .85 | .94 | 1.03 | 1.13 | 1.22 | 1.32 | 1.42 | 1.52 |
| .75 | .59 | .68 | .77 | .86 | .95 | 1.04 | 1.14 | 1.24 | 1.34 | 1.44 |
| .50 | .51 | .60 | .69 | .78 | .87 | .96 | 1.06 | 1.16 | 1.26 | 1.36 |
| .25 | .43 | .51 | .60 | .69 | .79 | .88 | .97 | 1.07 | 1.17 | 1.28 |
| 0.00 | .35 | .43 | .52 | .61 | .71 | .80 | .90 | 1.00 | 1.10 | 1.20 |
| | 8.00 | 10.00 | 12.00 | 14.00 | 16.00 | 18.00 | 20.00 | 22.00 | 24.00 | 26.00 P LAYER 1 |

TABLE 6.2c

Stair Hole Stability Analysis - phreatic surface artesian at slope crest

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 8.45 | 2: | 15.00 | 8.95 | 3: | 39.15 | 11.55 | 4: | 40.80 | 13.45 |
| 5: | 43.55 | 14.60 | 6: | 46.45 | 14.80 | 7: | 60.55 | 18.75 | 8: | 71.95 | 21.70 |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 4.94 | 3.68 | .32 | 11.40 | 0.00-2.25 | 26.00-8.00 |
| 2 | 14.66 | 6.83 | .27 | 17.00 | 0.00-2.25 | 26.00-8.00 |
| 3 | 18.41 | 7.27 | .19 | 14.40 | 0.00-2.25 | 26.00-8.00 |
| 4 | 20.73 | 8.58 | .18 | 14.15 | 0.00-2.25 | 26.00-8.00 |
| 5 | 6.39 | 7.11 | .09 | 2.70 | 0.00-2.25 | 26.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 8.01 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | 1.08 | 1.17 | 1.25 | 1.34 | 1.43 | 1.52 | 1.62 | 1.71 | 1.81 | 1.91 |
| 2.00 | 1.00 | 1.08 | 1.17 | 1.26 | 1.35 | 1.44 | 1.53 | 1.63 | 1.73 | 1.83 |
| 1.75 | .91 | 1.00 | 1.09 | 1.18 | 1.27 | 1.36 | 1.45 | 1.55 | 1.65 | 1.75 |
| 1.50 | .83 | .92 | 1.01 | 1.09 | 1.18 | 1.28 | 1.37 | 1.47 | 1.56 | 1.67 |
| 1.25 | .75 | .84 | .92 | 1.01 | 1.10 | 1.19 | 1.29 | 1.38 | 1.48 | 1.58 |
| 1.00 | .67 | .75 | .84 | .93 | 1.02 | 1.11 | 1.20 | 1.30 | 1.40 | 1.50 |
| .75 | .59 | .67 | .76 | .85 | .94 | 1.03 | 1.12 | 1.22 | 1.32 | 1.42 |
| .50 | .50 | .59 | .68 | .76 | .85 | .95 | 1.04 | 1.14 | 1.24 | 1.34 |
| .25 | .42 | .51 | .59 | .68 | .77 | .86 | .96 | 1.06 | 1.16 | 1.26 |
| 0.00 | .34 | .43 | .51 | .60 | .69 | .79 | .88 | .98 | 1.08 | 1.18 |
| | 8.00 | 10.00 | 12.00 | 14.00 | 16.00 | 18.00 | 20.00 | 22.00 | 24.00 | 26.00 P LAYER 1 |

TABLE 6.2d

Stair Hole Stability Analysis - phreatic surface 0.1 m higher

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 8.55 | 2: | 15.00 | 9.05 | 3: | 29.15 | 11.65 | 4: | 40.80 | 13.55 |
| 5: | 43.55 | 14.70 | 6: | 46.45 | 14.90 | 7: | 60.55 | 18.85 | 8: | 71.95 | 21.80 |

HORIZONTAL FORCE AT REAR (Q) = 0.05 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 4.94 | 4.66 | .32 | 11.40 | 0.00-2.25 | 26.00-8.00 |
| 2 | 14.66 | 7.81 | .27 | 17.00 | 0.00-2.25 | 26.00-8.00 |
| 3 | 18.41 | 8.25 | .19 | 14.40 | 0.00-2.25 | 26.00-8.00 |
| 4 | 20.73 | 9.56 | .18 | 14.15 | 0.00-2.25 | 26.00-8.00 |
| 5 | 6.39 | 8.09 | .09 | 2.70 | 0.00-2.25 | 26.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 7.03 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | 1.03 | 1.11 | 1.18 | 1.26 | 1.34 | 1.42 | 1.50 | 1.58 | 1.67 | 1.76 |
| 2.00 | .95 | 1.03 | 1.10 | 1.18 | 1.26 | 1.33 | 1.42 | 1.50 | 1.59 | 1.67 |
| 1.75 | .87 | .94 | 1.02 | 1.10 | 1.17 | 1.25 | 1.33 | 1.42 | 1.50 | 1.59 |
| 1.50 | .79 | .86 | .94 | 1.01 | 1.09 | 1.17 | 1.25 | 1.33 | 1.42 | 1.51 |
| 1.25 | .70 | .78 | .85 | .93 | 1.01 | 1.09 | 1.17 | 1.25 | 1.34 | 1.43 |
| 1.00 | .62 | .70 | .77 | .85 | .93 | 1.01 | 1.09 | 1.17 | 1.26 | 1.35 |
| .75 | .54 | .61 | .69 | .77 | .84 | .92 | 1.01 | 1.09 | 1.17 | 1.26 |
| .50 | .46 | .53 | .61 | .68 | .76 | .84 | .92 | 1.01 | 1.10 | 1.19 |
| .25 | .38 | .45 | .53 | .60 | .68 | .76 | .85 | .93 | 1.02 | 1.11 |
| 0.00 | .30 | .37 | .45 | .53 | .61 | .69 | .77 | .85 | .94 | 1.03 |
| | 8.00 | 10.00 | 12.00 | 14.00 | 16.00 | 18.00 | 20.00 | 22.00 | 24.00 | 26.00 P LAYER 1 |

TABLE 6.2e

Stair Hole Stability Analysis - phreatic surface 0.25 m higher

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 8.70 | 2: | 15.00 | 9.30 | 3: | 29.15 | 11.80 | 4: | 40.80 | 13.70 |
| 5: | 43.55 | 14.85 | 6: | 46.45 | 20.05 | 7: | 60.55 | 19.00 | 8: | 71.95 | 21.95 |

HORIZONTAL FORCE AT REAR (Q) = 0.31 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 4.94 | 6.13 | .32 | 11.40 | 0.00-2.25 | 26.00-8.00 |
| 2 | 14.66 | 33.80 | .27 | 17.00 | 0.00-2.25 | 26.00-8.00 |
| 3 | 18.41 | 9.72 | .19 | 14.40 | 0.00-2.25 | 26.00-8.00 |
| 4 | 20.73 | 11.03 | .18 | 14.15 | 0.00-2.25 | 26.00-8.00 |
| 5 | 6.39 | 9.56 | .09 | 2.70 | 0.00-2.25 | 26.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = -1.43 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | .70 | .74 | .78 | .82 | .87 | .92 | .96 | 1.01 | 1.06 | 1.11 |
| 2.00 | .64 | .68 | .72 | .77 | .81 | .86 | .90 | .95 | 1.00 | 1.05 |
| 1.75 | .58 | .62 | .66 | .71 | .75 | .80 | .84 | .89 | .94 | .99 |
| 1.50 | .52 | .56 | .61 | .65 | .69 | .74 | .79 | .83 | .88 | .93 |
| 1.25 | .46 | .50 | .55 | .59 | .64 | .68 | .73 | .78 | .83 | .88 |
| 1.00 | .40 | .44 | .49 | .53 | .58 | .62 | .67 | .72 | .77 | .83 |
| .75 | .34 | .39 | .43 | .47 | .52 | .57 | .62 | .67 | .72 | .77 |
| .50 | .29 | .33 | .37 | .42 | .47 | .51 | .57 | .62 | .68 | .73 |
| .25 | .23 | .00 | .32 | .37 | .42 | .48 | .53 | .58 | .64 | .69 |
| 0.00 | .19 | .24 | .29 | .33 | .39 | .44 | .49 | .54 | .60 | .65 |
| | 8.00 | 10.00 | 12.00 | 14.00 | 16.00 | 18.00 | 20.00 | 22.00 | 24.00 | 26.00 P LAYER 1 |

TABLE 6.2f

Stair Hole Stability Analysis - phreatic surface 0.5 m higher

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 8.95 | 2: | 15.00 | 9.45 | 3: | 29.15 | 12.05 | 4: | 40.80 | 13.95 |
| 5: | 43.55 | 15.10 | 6: | 46.45 | 15.30 | 7: | 60.55 | 19.25 | 8: | 71.95 | 22.20 |

HORIZONTAL FORCE AT REAR (Q) = 1.23 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 4.94 | 8.58 | .32 | 11.40 | 0.00-2.25 | 26.00-8.00 |
| 2 | 14.66 | 11.73 | .27 | 17.00 | 0.00-2.25 | 26.00-8.00 |
| 3 | 18.41 | 12.17 | .19 | 14.40 | 0.00-2.25 | 26.00-8.00 |
| 4 | 20.73 | 13.48 | .18 | 14.15 | 0.00-2.25 | 26.00-8.00 |
| 5 | 6.39 | 12.01 | .09 | 2.70 | 0.00-2.25 | 26.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 3.11 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | .85 | .87 | .90 | .93 | .96 | .99 | 1.02 | 1.05 | 1.09 | 1.13 |
| 2.00 | .76 | .79 | .82 | .85 | .88 | .91 | .94 | .98 | 1.01 | 1.05 |
| 1.75 | .68 | .71 | .74 | .77 | .80 | .83 | .86 | .90 | .93 | .99 |
| 1.50 | .60 | .63 | .66 | .69 | .72 | .75 | .79 | .83 | .88 | .93 |
| 1.25 | .52 | .55 | .58 | .61 | .64 | .67 | .73 | .77 | .82 | .87 |
| 1.00 | .44 | .47 | .50 | .53 | .57 | .62 | .66 | .71 | .76 | .81 |
| .75 | .36 | .39 | .42 | .46 | .50 | .56 | .60 | .65 | .70 | .74 |
| .50 | .28 | .32 | .36 | .40 | .45 | .50 | .54 | .59 | .63 | .68 |
| .25 | .22 | .26 | .30 | .34 | .39 | .43 | .48 | .52 | .57 | .62 |
| 0.00 | .16 | .20 | .24 | .28 | .33 | .37 | .42 | .46 | .51 | .56 |
| | 8.00 | 10.00 | 12.00 | 14.00 | 16.00 | 18.00 | 20.00 | 22.00 | 24.00 | 26.00 P LAYER 1 |

TABLE 6.2g

Stair Hole Stability Analysis - phreatic surface 0.1 m lower

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 8.35 | 2: | 15.00 | 8.85 | 3: | 29.15 | 11.45 | 4: | 40.80 | 13.35 |
| 5: | 43.55 | 14.50 | 6: | 46.45 | 14.70 | 7: | 60.55 | 18.65 | 8: | 71.95 | 21.60 |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 4.94 | 2.76 | .32 | 11.40 | 0.00-2.25 | 26.00-8.00 |
| 2 | 14.66 | 5.85 | .27 | 17.00 | 0.00-2.25 | 26.00-8.00 |
| 3 | 18.41 | 6.29 | .19 | 14.40 | 0.00-2.25 | 26.00-8.00 |
| 4 | 20.73 | 7.60 | .18 | 14.15 | 0.00-2.25 | 26.00-8.00 |
| 5 | 6.39 | 6.13 | .09 | 2.70 | 0.00-2.25 | 26.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 8.98 $kNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | 1.12 | 1.22 | 1.32 | 1.42 | 1.52 | 1.63 | 1.73 | 1.84 | 1.95 | 2.07 |
| 2.00 | 1.04 | 1.14 | 1.24 | 1.34 | 1.44 | 1.54 | 1.65 | 1.76 | 1.87 | 1.99 |
| 1.75 | .96 | 1.06 | 1.16 | 1.26 | 1.36 | 1.46 | 1.57 | 1.68 | 1.79 | 1.91 |
| 1.50 | .88 | .97 | 1.07 | 1.17 | 1.27 | 1.38 | 1.49 | 1.59 | 1.71 | 1.82 |
| 1.25 | .79 | .89 | .99 | 1.09 | 1.19 | 1.30 | 1.40 | 1.51 | 1.62 | 1.74 |
| 1.00 | .71 | .81 | .91 | 1.01 | 1.11 | 1.21 | 1.32 | 1.43 | 1.54 | 1.66 |
| .75 | .63 | .73 | .83 | .93 | 1.03 | 1.13 | 1.24 | 1.35 | 1.46 | 1.58 |
| .50 | .55 | .65 | .74 | .84 | .95 | 1.05 | 1.16 | 1.27 | 1.38 | 1.49 |
| .25 | .47 | .56 | .66 | .76 | .86 | .97 | 1.07 | 1.18 | 1.30 | 1.41 |
| 0.00 | .38 | .48 | .58 | .68 | .78 | .89 | .99 | 1.10 | 1.21 | 1.33 |
| | 8.00 | 10.00 | 12.00 | 14.00 | 16.00 | 18.00 | 20.00 | 22.00 | 24.00 | 26.00 P LAYER 1 |

TABLE 6.2h

Stair Hole Stability Analysis - phreatic surface 0.25 m lower

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 12.30 | 8.30 | 2: | 15.00 | 8.70 | 3: | 29.15 | 11.30 | 4: | 40.80 | 13.20 |
| 5: | 43.55 | 14.35 | 6: | 46.45 | 14.55 | 7: | 60.55 | 18.50 | 8: | 71.95 | 21.45 |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 4.94 | 1.63 | .32 | 11.40 | 0.00-2.25 | 26.00-8.00 |
| 2 | 14.66 | 4.37 | .27 | 17.00 | 0.00-2.25 | 26.00-8.00 |
| 3 | 18.41 | 4.82 | .19 | 14.40 | 0.00-2.25 | 26.00-8.00 |
| 4 | 20.73 | 6.13 | .18 | 14.15 | 0.00-2.25 | 26.00-8.00 |
| 5 | 6.39 | 4.66 | .09 | 2.70 | 0.00-2.25 | 26.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 10.39 $kNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | 1.19 | 1.30 | 1.42 | 1.54 | 1.66 | 1.78 | 1.90 | 2.03 | 2.16 | 2.29 |
| 2.00 | 1.11 | 1.22 | 1.34 | 1.45 | 1.57 | 1.69 | 1.82 | 1.95 | 2.08 | 2.21 |
| 1.75 | 1.02 | 1.14 | 1.25 | 1.37 | 1.49 | 1.61 | 1.74 | 1.86 | 1.99 | 2.13 |
| 1.50 | .94 | 1.06 | 1.17 | 1.29 | 1.41 | 1.53 | 1.65 | 1.78 | 1.91 | 2.05 |
| 1.25 | .86 | .97 | 1.09 | 1.21 | 1.33 | 1.45 | 1.57 | 1.70 | 1.83 | 1.97 |
| 1.00 | .78 | .89 | 1.01 | 1.12 | 1.24 | 1.36 | 1.49 | 1.62 | 1.75 | 1.88 |
| .75 | .69 | .81 | .92 | 1.04 | 1.16 | 1.28 | 1.41 | 1.53 | 1.67 | 1.80 |
| .50 | .61 | .73 | .84 | .96 | 1.08 | 1.20 | 1.32 | 1.45 | 1.58 | 1.72 |
| .25 | .53 | .64 | .76 | .88 | 1.00 | 1.12 | 1.24 | 1.37 | 1.50 | 1.64 |
| 0.00 | .45 | .56 | .68 | .79 | .91 | 1.04 | 1.16 | 1.29 | 1.42 | 1.55 |
| | 8.00 | 10.00 | 12.00 | 14.00 | 16.00 | 18.00 | 20.00 | 22.00 | 24.00 | 26.00 P LAYER 1 |

the same co-ordinate point but the phreatic surface was assumed to lie at some point above this.

At Mupe Bay (Figure 6.3, Table 6.3) the ground surface divides into 19, 30 m sections, with the slip surface almost mirroring the ground profile, particularly towards the toe of the mudslide. Only at the top of the slip does the depth of the mudslide increase, due to the recent addition of material from the head scarp. When compared with other sites this slope profile is steep throughout its length, with no significant flattening out at either the head or toe. Much care was therefore necessary in identifying the rear of this mudslide. A previously active slip covering the entire length of the slope is present, with the current movement appearing to result from regeneration of this relict feature. Consequently, for the purpose of stability analysis, a point approximately two thirds of the way upslope was identified as the current backscar. The mudslide at Mupe is very shallow and some difficulty was experienced in measuring the depth of the phreatic surface, both because movement frequently opened fissures likely to cause a drop in pore water pressure and also due to the length of the ceramic tip of the piezometer. The highest pore water pressures were monitored during January and February 1984. On these occasions one section of the slip surface remained exposed, with argillaceous material being 'rafted' over it at intervals. In many ways this is similar in character to the toe slope mudslide section at Stair Hole, except that at Mupe, this occurs in the centre of the mudslide and not at the toe, and the toe slope section is only laid bare during particularly wet conditions. At Stair Hole, the toe slope section remains uncovered for most of the year. In addition at Mupe, the whole mudslide profile is within the Wealden Beds, while at Stair

TABLE 6.3a

Mupe Bay Stability Analysis - parameter co-ordinates

Ground Surface (Layer Beneath in Brackets)

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | 0.00 | (1) | 2: | 2.10 | 1.95 | (1) | 3: | 4.80 | 3.20 | (1) |
| 4: | 7.45 | 1.40 | (1) | 5: | 9.85 | 6.05 | (1) | 6: | 12.35 | 7.75 | (1) |
| 7: | 14.50 | 9.50 | (1) | 8: | 16.80 | 11.30 | (1) | 9: | 19.30 | 12.30 | (1) |
| 10: | 21.95 | 14.05 | (1) | 11: | 24.55 | 15.30 | (1) | 12: | 27.20 | 16.40 | (1) |
| 13: | 30.00 | 17.35 | (1) | 14: | 32.55 | 18.75 | (1) | 15: | 35.00 | 20.25 | (1) |
| 16: | 37.30 | 22.05 | (1) | 17: | 39.55 | 23.90 | (1) | 18: | 42.25 | 25.00 | (1) |
| 19: | 44.90 | 26.30 | (1) | | | | | | | | |

Strength Parameters

| | PRIMARY | | | SECONDARY | | | |
|---|---|---|---|---|---|---|---|
| | COHESION (kPa) | PHI | DENSITY ($Mgm^{-3}$) | COHESION (kPa) | PHI | IN RANGE LOWER | UPPER |
| LAYER 1: | 0.00 | 22.00 | 1.76 | --- | --- | --- | --- |

Failure Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | 0.00 | 2: | 4.80 | 2.40 | 3: | 7.45 | 3.65 | 4: | 16.80 | 10.75 |
| 5: | 27.20 | 15.70 | 6: | 30.00 | 16.60 | 7: | 42.25 | 24.35 | 8: | 44.90 | 26.30 |

TABLE 6.3b

Mupe Bay Stability Analysis - phreatic surface as measured

HORIZONTAL FORCE AT REAR (Q) = .11 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 13.24 | 2.74 | .63 | 13.50 | 0.00-2.25 | 40.00-22.00 |
| 2 | 12.55 | 4.41 | .32 | 2.80 | 0.00-2.25 | 40.00-22.00 |
| 3 | 13.85 | 3.43 | .48 | 10.40 | 0.00-2.25 | 40.00-22.00 |
| 4 | 4.76 | .61 | .54 | 2.30 | 0.00-2.25 | 40.00-22.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 9.74 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | .95 | 1.01 | 1.06 | 1.12 | 1.18 | 1.24 | 1.31 | 1.38 | 1.45 | 1.53 |
| 2.00 | .91 | .96 | 1.02 | 1.08 | 1.14 | 1.20 | 1.26 | 1.33 | 1.40 | 1.48 |
| 1.75 | .86 | .91 | .97 | 1.03 | 1.09 | 1.15 | 1.22 | 1.29 | 1.36 | 1.43 |
| 1.50 | .81 | .87 | .92 | .98 | 1.04 | 1.10 | 1.17 | 1.24 | 1.31 | 1.39 |
| 1.25 | .76 | .82 | .88 | .94 | 1.00 | 1.06 | 1.12 | 1.19 | 1.26 | 1.34 |
| 1.00 | .72 | .77 | .83 | .89 | .95 | 1.01 | 1.08 | 1.14 | 1.22 | 1.29 |
| .75 | .67 | .73 | .78 | .84 | .90 | .96 | 1.03 | 1.10 | 1.17 | 1.24 |
| .50 | .63 | .68 | .74 | .79 | .85 | .92 | .98 | 1.05 | 1.12 | 1.20 |
| .25 | .58 | .63 | .69 | .75 | .81 | .87 | .93 | 1.00 | 1.07 | 1.15 |
| 0.00 | .53 | .59 | .64 | .70 | .76 | .82 | .89 | .95 | 1.03 | 1.10 |
| | 22.00 | 24.00 | 26.00 | 28.00 | 30.00 | 32.00 | 34.00 | 36.00 | 38.00 | 40.00 P LAYER 1 |

TABLE 6.3c

Mupe Bay Stability Analysis - phreatic surface 0.1 m lower

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 14.50 | 9.15 | 2: | 16.80 | 10.90 | 3: | 27.20 | 16.05 | 4: | 30.00 | 16.95 |
| 5: | 43.25 | 24.35 | 6: | 44.90 | 26.05 | | | | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.01 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | SHEAR STRENGTH PHI |
|---|---|---|---|---|---|---|
| 1 | 13.34 | 1.76 | .63 | 13.50 | 0.00-2.25 | 40.00-22.00 |
| 2 | 12.55 | 3.43 | .82 | 2.80 | 0.00-2.25 | 40.00-22.00 |
| 3 | 13.85 | 2.45 | .48 | 10.40 | 0.00-2.25 | 40.00-22.00 |
| 4 | 4.76 | .22 | .54 | 2.30 | 0.00-2.25 | 40.00-22.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 10.67 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | 1.03 | 1.09 | 1.15 | 1.22 | 1.28 | 1.36 | 1.43 | 1.51 | 1.59 | 1.67 |
| 2.00 | .98 | 1.04 | 1.10 | 1.17 | 1.24 | 1.31 | 1.38 | 1.46 | 1.54 | 1.63 |
| 1.75 | .93 | .99 | 1.06 | 1.12 | 1.19 | 1.26 | 1.34 | 1.41 | 1.49 | 1.58 |
| 1.50 | .88 | .95 | 1.01 | 1.08 | 1.14 | 1.21 | 1.29 | 1.37 | 1.45 | 1.53 |
| 1.25 | .84 | .90 | .96 | 1.03 | 1.10 | 1.17 | 1.24 | 1.32 | 1.40 | 1.49 |
| 1.00 | .79 | .85 | .91 | .98 | 1.05 | 1.12 | 1.20 | 1.27 | 1.36 | 1.44 |
| .75 | .74 | .81 | .87 | .94 | 1.00 | 1.08 | 1.15 | 1.23 | 1.31 | 1.39 |
| .50 | .70 | .76 | .82 | .89 | .96 | 1.03 | 1.10 | 1.18 | 1.26 | 1.35 |
| .25 | .65 | .71 | .78 | .84 | .91 | .98 | 1.05 | 1.13 | 1.21 | 1.30 |
| 0.00 | .60 | .66 | .73 | .79 | .86 | .93 | 1.01 | 1.09 | 1.16 | 1.25 |
| | 22.00 | 24.00 | 26.00 | 28.00 | 30.00 | 32.00 | 34.00 | 36.00 | 38.00 | 40.00 P LAYER 1 |

TABLE 6.3d

Mupe Bay Stability Analysis - phreatic surface 0.25 m lower

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 14.50 | 9.00 | 2: | 16.80 | 10.75 | 3: | 27.20 | 15.90 | 4: | 30.00 | 16.80 |
| 5: | 42.25 | 24.20 | 6: | 44.90 | 25.95 | | | | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 13.24 | .57 | .63 | 13.50 | 0.00-2.25 | 40.00-22.00 |
| 2 | 12.55 | 1.96 | .32 | 2.80 | 0.00-2.25 | 40.00-22.00 |
| 3 | 13.85 | .98 | .48 | 10.40 | 0.00-2.25 | 40.00-22.00 |
| 4 | 4.76 | .00 | .54 | 2.30 | 0.00-2.25 | 40.00-22.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 11.91 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | 1.12 | 1.19 | 1.27 | 1.34 | 1.42 | 1.50 | 1.59 | 1.68 | 1.77 | 1.88 |
| 2.00 | 1.08 | 1.15 | 1.22 | 1.30 | 1.37 | 1.46 | 1.54 | 1.63 | 1.73 | 1.84 |
| 1.75 | 1.03 | 1.10 | 1.17 | 1.25 | 1.33 | 1.41 | 1.49 | 1.58 | 1.68 | 1.78 |
| 1.50 | .98 | 1.05 | 1.12 | 1.20 | 1.28 | 1.36 | 1.45 | 1.55 | 1.64 | 1.74 |
| 1.25 | .93 | 1.00 | 1.08 | 1.15 | 1.23 | 1.31 | 1.40 | 1.49 | 1.59 | 1.69 |
| 1.00 | .88 | .96 | 1.03 | 1.11 | 1.18 | 1.27 | 1.35 | 1.44 | 1.54 | 1.65 |
| .75 | .84 | .91 | .98 | 1.06 | 1.14 | 1.22 | 1.30 | 1.39 | 1.49 | 1.59 |
| .50 | .79 | .86 | .94 | 1.01 | 1.09 | 1.17 | 1.26 | 1.35 | 1.44 | 1.54 |
| .25 | .74 | .81 | .89 | .96 | 1.04 | 1.12 | 1.21 | 1.30 | 1.40 | 1.50 |
| 0.00 | .70 | .77 | .84 | .92 | 1.00 | 1.08 | 1.17 | 1.26 | 1.35 | 1.45 |
| | 22.00 | 24.00 | 26.00 | 28.00 | 30.00 | 32.00 | 34.00 | 36.00 | 38.00 | 40.00 P LAYER 1 |

TABLE 6.3e

Mupe Bay Stability Analysis - phreatic surface 0.1 m higher

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 14.50 | 9.35 | 2: | 16.80 | 11.10 | 3: | 27.20 | 16.25 | 4: | 30.00 | 17.15 |
| 5: | 42.25 | 24.55 | 6: | 44.90 | 26.25 | | | | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.31 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 13.24 | 3.72 | .63 | 13.50 | 0.00-2.25 | 40.00-22.00 |
| 2 | 12.55 | 5.39 | .32 | 2.80 | 0.00-2.25 | 40.00-22.00 |
| 3 | 13.85 | 4.41 | .48 | 10.40 | 0.00-2.25 | 40.00-22.00 |
| 4 | 4.76 | 1.20 | .54 | 2.30 | 0.00-2.25 | 40.00-22.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 8.79 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | .88 | .93 | .98 | 1.03 | 1.08 | 1.13 | 1.19 | 1.25 | 1.31 | 1.38 |
| 2.00 | .83 | .88 | .93 | .98 | 1.03 | 1.09 | 1.14 | 1.20 | 1.26 | 1.33 |
| 1.75 | .79 | .83 | .88 | .93 | .99 | 1.04 | 1.09 | 1.15 | 1.22 | 1.28 |
| 1.50 | .74 | .79 | .84 | .89 | .94 | .99 | 1.05 | 1.11 | 1.17 | 1.23 |
| 1.25 | .69 | .74 | .79 | .84 | .89 | .94 | 1.00 | 1.06 | 1.12 | 1.19 |
| 1.00 | .65 | .69 | .74 | .79 | .84 | .90 | .95 | 1.01 | 1.07 | 1.14 |
| .75 | .60 | .65 | .70 | .75 | .79 | .85 | .90 | .96 | 1.03 | 1.09 |
| .50 | .55 | .60 | .65 | .70 | .75 | .80 | .86 | .92 | .98 | 1.04 |
| .25 | .50 | .55 | .60 | .65 | .70 | .75 | .81 | .87 | .93 | 1.00 |
| 0.00 | .46 | .50 | .55 | .60 | .65 | .71 | .76 | .82 | .89 | .95 |
| | 22.00 | 24.00 | 26.00 | 28.00 | 30.00 | 32.00 | 34.00 | 36.00 | 38.00 | 40.00 P LAYER 1 |

TABLE 6.3f

Mupe Bay Stability Analysis - phreatic surface 0.25 m higher

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 14.50 | 9.50 | 2: | 16.80 | 11.25 | 3: | 27.20 | 16.40 | 4: | 30.00 | 17.30 |
| 5: | 42.25 | 24.70 | 6: | 44.90 | 26.40 | | | | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.79 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 13.24 | 5.19 | .63 | 13.50 | 0.00-2.25 | 40.00-22.00 |
| 2 | 12.55 | 6.86 | .32 | 2.80 | 0.00-2.25 | 40.00-22.00 |
| 3 | 13.85 | 5.88 | .48 | 10.40 | 0.00-2.25 | 40.00-22.00 |
| 4 | 4.76 | 2.45 | .54 | 2.30 | 0.00-2.25 | 40.00-22.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 7.33 KNm$^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.25 | .77 | .80 | .84 | .88 | .92 | .96 | 1.00 | 1.04 | 1.09 | 1.14 |
| 2.00 | .72 | .76 | .79 | .83 | .87 | .91 | .95 | 1.00 | 1.04 | 1.09 |
| 1.75 | .68 | .71 | .75 | .79 | .83 | .86 | .91 | .95 | 1.00 | 1.05 |
| 1.50 | .63 | .66 | .70 | .74 | .78 | .82 | .86 | .90 | .95 | 1.00 |
| 1.25 | .58 | .62 | .65 | .69 | .73 | .77 | .81 | .86 | .90 | .95 |
| 1.00 | .54 | .57 | .61 | .64 | .68 | .72 | .77 | .81 | .86 | .91 |
| .75 | .49 | .52 | .56 | .60 | .64 | .68 | .72 | .76 | .81 | .86 |
| .50 | .44 | .48 | .51 | .55 | .59 | .63 | .67 | .72 | .76 | .81 |
| .25 | .39 | .43 | .47 | .50 | .54 | .58 | .63 | .67 | .71 | .76 |
| 0.00 | .35 | .38 | .42 | .46 | .50 | .53 | .58 | .62 | .67 | .72 |
| | 22.00 | 24.00 | 26.00 | 28.00 | 30.00 | 32.00 | 34.00 | 36.00 | 38.00 | 40.00 P LAYER 1 |

Hole, remnants of the Purbeck are responsible for the steep change in slope angle. Stability analysis was consequently conducted assuming total continuity of the mudslide mass.

Worbarrow Bay profile (Figure 6.4, Table 6.4) displays a slip surface which closely mirrors the ground surface, but gradually approaches the mudslide surface towards the rear of the slide. The piezometer installed at the toe of the slide is full throughout the year, despite the fact that drainage is into beach material, suggesting a particularly high phreatic surface in winter months.

### 6.3.2 Results and Discussion

The stability analyses are based on parameters previously discussed (section 6.2.1). It would detract from this study to present one specific Factor of Safety for each slope since all measured parameters are open to a degree of variance, making a single result unjustifiable relative to the data. The parameters vary through time and space and therefore a range of results is of greater significance. In addition, the presentation of a variety of Factors of Safety bears greater relevance to other parts of this study, particularly the geotechnical analysis (chapter IV) and field monitoring (chapter V). Consequently for each site a Factor of Safety matrix is presented. These results can be combined with field and laboratory measurements to make assumptions and reach important conclusions regarding slope characteristics.

TABLE 6.4a

Worbarrow Bay Stability Analysis - parameter co-ordinates

Ground Surface (Layer Beneath in Brackets)

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | 0.00 | (1) | 2: | 2.75 | 1.30 | (1) | 3: | 5.60 | 2.15 | (1) |
| 4: | 8.45 | 2.80 | (1) | 5: | 11.40 | 3.35 | (1) | 6: | 14.30 | 3.90 | (1) |
| 7: | 17.10 | 4.75 | (1) | 8: | 19.90 | 5.50 | (1) | 9: | 22.70 | 6.90 | (1) |
| 10: | 25.35 | 8.30 | (1) | 11: | 29.00 | 9.55 | (1) | 12: | 30.70 | 10.85 | (1) |
| 13: | 33.40 | 12.10 | (1) | 14: | 36.05 | 13.45 | (1) | 15: | 38.80 | 14.50 | (1) |
| 16: | 41.50 | 15.80 | (1) | 17: | 44.05 | 17.70 | (1) | | | | |

Strength Parameters

| | PRIMARY | | | SECONDARY | | | |
|---|---|---|---|---|---|---|---|
| | COHESION (kPa) | PHI | DENSITY ($Mgm^{-3}$) | COHESION (kPa) | PHI | IN RANGE LOWER | IN RANGE UPPER |
| LAYER 1: | 0.00 | 22.00 | 1.72 | --- | --- | --- | --- |

Failure Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | 0.00 | 2: | 8.45 | 1.85 | 3: | 17.10 | 3.70 | 4: | 22.70 | 5.90 |
| 5: | 25.35 | 7.20 | 6: | 33.40 | 11.35 | 7: | 41.50 | 15.35 | 8: | 44.05 | 17.20 |

TABLE 6.4b

Worbarrow Bay Stability Analysis - phreatic surface as measured

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | 0.00 | 2: | 2.75 | 2.20 | 3: | 8.45 | 2.80 | 4: | 17.10 | 4.65 |
| 5: | 25.35 | 7.90 | 6: | 41.50 | 15.80 | 7: | 44.05 | 17.20 | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 3.80 | 2.21 | .73 | 2.55 | 0.00-4.50 | 44.00-8.00 |
| 2 | 10.53 | 4.60 | .49 | 8.10 | 0.00-4.50 | 44.00-8.00 |
| 3 | 15.46 | 5.82 | .52 | 8.05 | 0.00-4.50 | 44.00-8.00 |
| 4 | 16.87 | 8.12 | .49 | 2.65 | 0.00-4.50 | 44.00-8.00 |
| 5 | 17.08 | 9.35 | .39 | 5.60 | 0.00-4.50 | 44.00-8.00 |
| 6 | 15.00 | 9.32 | .21 | 8.65 | 0.00-4.50 | 44.00-8.00 |
| 7 | 11.86 | 10.98 | .22 | 8.45 | 0.00-4.50 | 44.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 5.74 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 4.50 | 1.18 | 1.25 | 1.32 | 1.39 | 1.47 | 1.55 | 1.64 | 1.73 | 1.84 | 1.96 |
| 4.00 | 1.06 | 1.13 | 1.20 | 1.28 | 1.35 | 1.43 | 1.52 | 1.62 | 1.72 | 1.84 |
| 3.50 | .95 | 1.02 | 1.09 | 1.16 | 1.24 | 1.32 | 1.40 | 1.50 | 1.61 | 1.73 |
| 3.00 | .83 | .90 | .97 | 1.04 | 1.12 | 1.20 | 1.29 | 1.38 | 1.49 | 1.61 |
| 2.50 | .72 | .78 | .85 | .93 | 1.00 | 1.08 | 1.17 | 1.27 | 1.37 | 1.49 |
| 2.00 | .60 | .67 | .74 | .81 | .89 | .97 | 1.06 | 1.15 | 1.26 | 1.38 |
| 1.50 | .48 | .55 | .62 | .69 | .77 | .85 | .94 | 1.03 | 1.14 | 1.26 |
| 1.00 | .36 | .43 | .50 | .58 | .65 | .73 | .82 | .92 | 1.02 | 1.14 |
| .50 | .25 | .32 | .39 | .46 | .54 | .62 | .70 | .80 | .91 | 1.02 |
| 0.00 | .13 | .20 | .27 | .34 | .42 | .50 | .59 | .68 | .79 | .90 |
| | 8.00 | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 | 32.00 | 36.00 | 40.00 | 44.00 P LAYER 1 |

TABLE 6.4c

Worbarrow Bay Stability Analysis - phreatic surface 0.1 m higher

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | .10 | 2: | 2.75 | 2.30 | 3: | 8.45 | 2.90 | 4: | 17.10 | 4.75 |
| 5: | 25.35 | 8.00 | 6: | 41.50 | 15.90 | 7: | 44.05 | 17.30 | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.05 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 3.80 | 3.19 | .73 | 2.55 | 0.00-4.50 | 44.00-8.00 |
| 2 | 10.53 | 5.58 | .49 | 8.10 | 0.00-4.50 | 44.00-8.00 |
| 3 | 15.46 | 6.80 | .52 | 8.05 | 0.00-4.50 | 44.00-8.00 |
| 4 | 16.87 | 9.10 | .49 | 2.65 | 0.00-4.50 | 44.00-8.00 |
| 5 | 17.08 | 10.33 | .39 | 5.60 | 0.00-4.50 | 44.00-8.00 |
| 6 | 15.00 | 10.30 | .21 | 8.65 | 0.00-4.50 | 44.00-8.00 |
| 7 | 11.86 | 11.96 | .22 | 8.45 | 0.00-4.50 | 44.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 4.76 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 4.50 | 1.15 | 1.20 | 1.25 | 1.31 | 1.37 | 1.43 | 1.49 | 1.57 | 1.65 | 1.74 |
| 4.00 | 1.03 | 1.08 | 1.14 | 1.19 | 1.25 | 1.31 | 1.38 | 1.45 | 1.53 | 1.62 |
| 3.50 | .92 | .97 | 1.02 | 1.08 | 1.13 | 1.20 | 1.26 | 1.34 | 1.41 | 1.50 |
| 3.00 | .80 | .85 | .90 | .96 | 1.02 | 1.08 | 1.15 | 1.22 | 1.30 | 1.39 |
| 2.50 | .68 | .73 | .79 | .84 | .90 | .96 | 1.03 | 1.10 | 1.18 | 1.27 |
| 2.00 | .57 | .62 | .67 | .73 | .78 | .85 | .91 | .98 | 1.06 | 1.15 |
| 1.50 | .45 | .50 | .55 | .61 | .67 | .73 | .79 | .87 | .95 | 1.04 |
| 1.00 | .33 | .39 | .44 | .49 | .55 | .61 | .68 | .75 | .83 | .92 |
| .50 | .22 | .27 | .32 | .38 | .43 | .49 | .56 | .63 | .71 | .80 |
| 0.00 | .10 | .16 | .21 | .26 | .33 | .39 | .46 | .54 | .62 | .72 |
| | 8.00 | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 | 32.00 | 36.00 | 40.00 | 44.00 P LAYER 1 |

TABLE 6.4d

Worbarrow Bay Stability Analysis - phreatic surface 0.1 m lower

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | -.10 | 2: | 2.75 | 2.10 | 3: | 8.45 | 2.70 | 4: | 17.10 | 4.55 |
| 5: | 25.35 | 7.80 | 6: | 41.50 | 15.70 | 7: | 44.05 | 17.10 | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 3.80 | 1.33 | .73 | 2.55 | 0.00-4.50 | 44.00-8.00 |
| 2 | 10.53 | 3.62 | .49 | 8.10 | 0.00-4.50 | 44.00-8.00 |
| 3 | 15.46 | 4.84 | .52 | 8.05 | 0.00-4.50 | 44.00-8.00 |
| 4 | 16.87 | 7.14 | .49 | 2.65 | 0.00-4.50 | 44.00-8.00 |
| 5 | 17.08 | 8.37 | .39 | 5.60 | 0.00-4.50 | 44.00-8.00 |
| 6 | 15.00 | 8.34 | .21 | 8.65 | 0.00-4.50 | 44.00-8.00 |
| 7 | 11.86 | 10.01 | .22 | 8.45 | 0.00-4.50 | 44.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 6.71 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 4.50 | 1.21 | 1.30 | 1.38 | 1.47 | 1.57 | 1.67 | 1.79 | 1.91 | 2.04 | 2.19 |
| 4.00 | 1.10 | 1.18 | 1.27 | 1.36 | 1.45 | 1.55 | 1.66 | 1.78 | 1.92 | 2.06 |
| 3.50 | .98 | 1.06 | 1.15 | 1.24 | 1.34 | 1.44 | 1.55 | 1.66 | 1.80 | 1.95 |
| 3.00 | .86 | .95 | 1.03 | 1.12 | 1.22 | 1.32 | 1.43 | 1.55 | 1.68 | 1.83 |
| 2.50 | .75 | .83 | .92 | 1.01 | 1.10 | 1.20 | 1.32 | 1.43 | 1.56 | 1.71 |
| 2.00 | .63 | .71 | .80 | .89 | .99 | 1.09 | 1.20 | 1.32 | 1.45 | 1.59 |
| 1.50 | .51 | .60 | .68 | .78 | .87 | .97 | 1.08 | 1.20 | 1.33 | 1.48 |
| 1.00 | .40 | .48 | .57 | .66 | .75 | .86 | .96 | 1.08 | 1.21 | 1.36 |
| .50 | .28 | .37 | .45 | .54 | .64 | .74 | .85 | .96 | 1.10 | 1.24 |
| 0.00 | .17 | .25 | .34 | .42 | .52 | .62 | .73 | .85 | .98 | 1.13 |
| | 8.00 | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 | 32.00 | 36.00 | 40.00 | 44.00 P LAYER 1 |

TABLE 6.4e

Worbarrow Bay Stability Analysis - phreatic surface 0.25 m lower

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | -.25 | 2: | 2.75 | 1.95 | 3: | 8.45 | 2.55 | 4: | 17.10 | 4.40 |
| 5: | 25.35 | 7.65 | 6: | 41.50 | 15.55 | 7: | 44.05 | 16.95 | | | |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 3.80 | .44 | .73 | 2.55 | 0.00-4.50 | 44.00-8.00 |
| 2 | 10.53 | 2.15 | .49 | 8.10 | 0.00-4.50 | 44.00-8.00 |
| 3 | 15.46 | 3.37 | .52 | 8.05 | 0.00-4.50 | 44.00-8.00 |
| 4 | 16.87 | 5.67 | .49 | 2.65 | 0.00-4.50 | 44.00-8.00 |
| 5 | 17.08 | 6.89 | .39 | 5.60 | 0.00-4.50 | 44.00-8.00 |
| 6 | 15.00 | 6.86 | .21 | 8.65 | 0.00-4.50 | 44.00-8.00 |
| 7 | 11.86 | 8.59 | .22 | 8.45 | 0.00-4.50 | 44.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 8.14 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 4.50 | 1.26 | 1.37 | 1.48 | 1.59 | 1.73 | 1.86 | 2.00 | 2.15 | 2.32 | 2.51 |
| 4.00 | 1.14 | 1.25 | 1.36 | 1.48 | 1.60 | 1.74 | 1.88 | 2.03 | 2.20 | 2.40 |
| 3.50 | 1.03 | 1.13 | 1.25 | 1.36 | 1.48 | 1.61 | 1.76 | 1.92 | 2.09 | 2.28 |
| 3.00 | .91 | 1.02 | 1.13 | 1.25 | 1.37 | 1.50 | 1.64 | 1.80 | 1.97 | 2.16 |
| 2.50 | .79 | .90 | 1.01 | 1.13 | 1.25 | 1.38 | 1.52 | 1.68 | 1.84 | 2.03 |
| 2.00 | .68 | .79 | .90 | 1.01 | 1.13 | 1.26 | 1.40 | 1.56 | 1.73 | 1.92 |
| 1.50 | .56 | .67 | .78 | .90 | 1.02 | 1.15 | 1.29 | 1.44 | 1.61 | 1.80 |
| 1.00 | .44 | .55 | .66 | .78 | .90 | 1.03 | 1.17 | 1.32 | 1.49 | 1.68 |
| .50 | .33 | .44 | .55 | .66 | .79 | .92 | 1.06 | 1.21 | 1.38 | 1.57 |
| 0.00 | .21 | .32 | .43 | .55 | .67 | .80 | .94 | 1.09 | 1.26 | 1.45 |
| | 8.00 | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 | 32.00 | 36.00 | 40.00 | 44.00 P LAYER 1 |

TABLE 6.4f

Worbarrow Bay Stability Analysis - phreatic surface dry except for the slope

Phreatic Surface

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1: | 0.00 | 0.00 | 2: | 2.75 | 2.20 | 3: | 8.45 | 2.80 | 4: | 17.10 | 3.70 |

HORIZONTAL FORCE AT REAR (Q) = 0.00 kPa

SLICE PARAMETERS

| SLICE NUMBER | TOTAL PRESSURE (kPa) | WATER PRESSURE (kPa) | TAN ALPHA | SLICE WIDTH (m) | SHEAR STRENGTH COHESION (kPa) | PHI |
|---|---|---|---|---|---|---|
| 1 | 3.80 | 0.00 | .73 | 2.55 | 0.00-4.50 | 44.00-8.00 |
| 2 | 10.53 | 0.00 | .49 | 8.10 | 0.00-4.50 | 44.00-8.00 |
| 3 | 15.46 | 0.00 | .52 | 8.05 | 0.00-4.50 | 44.00-8.00 |
| 4 | 16.87 | 0.00 | .49 | 2.65 | 0.00-4.50 | 44.00-8.00 |
| 5 | 17.08 | 0.00 | .39 | 5.60 | 0.00-4.50 | 44.00-8.00 |
| 6 | 15.00 | 4.66 | .21 | 8.65 | 0.00-4.50 | 44.00-8.00 |
| 7 | 11.86 | 10.98 | .22 | 8.45 | 0.00-4.50 | 44.00-8.00 |

AVERAGE EFFECTIVE PRESSURE ON SLIP SURFACE = 10.37 $KNm^{-2}$

FACTORS OF SAFETY

C LAYER 1

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 4.50 | 1.33 | 1.48 | 1.63 | 1.79 | 1.95 | 2.13 | 2.32 | 2.53 | 2.75 | 3.01 |
| 4.00 | 1.22 | 1.36 | 1.51 | 1.67 | 1.84 | 2.01 | 2.20 | 2.41 | 2.64 | 2.89 |
| 3.50 | 1.10 | 1.25 | 1.40 | 1.55 | 1.72 | 1.90 | 2.09 | 2.29 | 2.52 | 2.78 |
| 3.00 | .98 | 1.13 | 1.28 | 1.44 | 1.60 | 1.78 | 1.97 | 2.17 | 2.40 | 2.66 |
| 2.50 | .87 | 1.01 | 1.16 | 1.32 | 1.49 | 1.66 | 1.85 | 2.06 | 2.28 | 2.54 |
| 2.00 | .75 | .90 | 1.05 | 1.20 | 1.37 | 1.54 | 1.73 | 1.94 | 2.17 | 2.42 |
| 1.50 | .64 | .78 | .93 | 1.09 | 1.25 | 1.43 | 1.62 | 1.82 | 2.05 | 2.31 |
| 1.00 | .52 | .66 | .81 | .97 | 1.14 | 1.31 | 1.50 | 1.71 | 1.93 | 2.19 |
| .50 | .40 | .55 | .70 | .85 | 1.02 | 1.19 | 1.38 | 1.59 | 1.82 | 2.07 |
| 0.00 | .29 | .43 | .58 | .74 | .90 | 1.08 | 1.27 | 1.47 | 1.70 | 1.96 |
| | 8.00 | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 | 32.00 | 36.00 | 40.00 | 44.00 P LAYER 1 |

6.3.2.1 Durdle Door

The computer accepted all input. It can therefore be assumed that the measured slope geometries are indicative of site conditions. Measured parameters were used as the initial input and Factors of Safety calculated.

Assuming zero cohesion at residual strength, a Factor of Safety of 1 is present with residual friction angles between 32-34$^{o}$. Laboratory tests suggest, however, a residual angle of internal friction of 29-30$^{o}$ while the mean measured field slope angle is 30$^{o}$. At this value the computed Factor of Safety = 0.92 suggests that the slope is unstable. Field and laboratory measurements show that the slope is very close to its critical angle. A number of reasons for this can be suggested. The program analysis may underestimate the stability of the slope because the measured values may not be entirely representative of overall mean site conditions. This may, in turn, be due to the extreme sampling difficulties experienced at this site (see chapter IV) and the particularly sandy, highly variable material. Although the material is at residual strength, cohesion may be significant. Craig (1981), for example, suggested from investigations of the mudslides at East County Antrim, Northern Ireland, that dry season pore water suction might influence slope stability by increasing strength across the slip surface by introducing an element of residual cohesion. This is possible at Durdle Door, since the argument is supported by the observation that the standpipe piezometers remain empty for a large part of the year. There may also be some error in measured slope angles, but this was kept to an accuracy of $\pm$ 0.5$^{o}$, and is therefore not likely to be particularly significant. Finally, irregularities across the slip surface or considerable interparticle friction may exist due to the sand content. It is most likely that a combination of these influences are responsible for the low Factor of Safety at zero cohesion, since the results of the geomorphological mapping, geotechnical analysis and slope monitoring all suggest that the mudslides at Durdle Door are stable for much of the year.

The level of the phreatic surface used in the analysis represents its highest monitored level. It cannot be assumed however, that it does not rise above these values. Consequently, further analysis was conducted using higher pore water pressure values. This was completed in two stages, a change in the input representing a rise of the phreatic surface of 0.1 m and a second modification to investigate the effects of a 0.2 m increase. Since field monitoring shows this slope to be particularly stable, particularly under conditions of reduced pore water pressure, it was not felt necessary to calculate Factors of Safety for a lower phreatic surface.

With a 0.1 m rise in pore water pressure, there is a significant drop in the Factor of Safety to 0.69 at residual friction angle of 29-30$^{o}$ and zero cohesion. The angle of friction has to be greater than 40$^{o}$ under these conditions before a Factor of Safety of unity is surpassed. In reality therefore, a phreatic surface this high is unlikely to exist, with the slope failing long before these values are reached. It can thus be suggested that the measured phreatic surface is close to the maximum conditions which are likely to occur. An increase in pore water pressure of a further 0.1 m mirrors these results, with a mean drop in the Factor of Safety by 0.1-0.2 under zero cohesion. The decrease in the Factor of Safety is not, however, as significant for this further increase in the phreatic surface as it is for the first rise, suggesting that the effects of rising pore water pressures become exponentially less pronounced.

6.3.2.2 Stair Hole

Input was acceptable for analysis, suggesting the assumed slope geometries and measured material parameters are representative of site conditions. Results are presented in Table 6.2. Using measured parameters and assuming cohesion to be zero at residual strength, a Factor of Safety of 1 is recorded at a residual friction angle of $22^{o}$. Residual friction angles (see chapter IV) were obtained by laboratory testing for a number of samples taken from this site and a mean value of $20^{o}$ recorded. The associated Factor of Safety = 0.90, indicating unstable conditions under measured values. This is reflected by field conditions (see chapter V) under the measured pore water pressure maxima and associated rapid slope movements, particularly towards the toe of the slide. It is interesting to note that as the angle of the slope varies along the mudslide, so does the stability of the material. These different conditions can be related to the mean residual friction angle. For example, at the toe of the slope the mean slope angle is $32^{o}$, almost $10^{o}$ higher than the angle of internal friction determined in the laboratory. Consequently, as is suggested from the stability analysis, in winter during conditions of high pore water pressure, material slides straight over this slope section, leaving an exposed slip surface (see chapter III). If a cohesional element is assumed to exist at residual strength, the Factor of Safety rapidly increases to more than 1. However, this situation cannot be supported from the results.

Following initial stability calculations the input data was modified. The phreatic surface was raised at the rear of the slope because on occasions the piezometer at the head of the mudslide is full, suggesting that pore water pressures may be artesian. Attempts can thus be made to model these conditions and assess their effects. Secondly, the level of the whole phreatic surface was raised to record changes in the stability of the material, since measured maxima cannot be assumed to be the highest pore water pressures which occur. It was also decided to conduct tests following an overall lowering of the phreatic surface, in an attempt to establish the point at which the transition between stability and instability occurs. Such analysis is more relevant to this side than at Durdle Door, due to the variation in field conditions and movement of the material on the slope (see chapter V).

With increased pore water pressure at the rear of the slide, zero residual cohesion and a residual friction angle of $20^{o}$ the Factor of Safety of the slope drops slightly, but only in the order of 0.01-0.02. A Factor of Safety of unity now requires a material residual friction angle of between $22\text{-}24^{o}$. Thus, although there is an influence on the overall slope stability, this is small. Results therefore suggest that a general rise in the phreatic surface is required across the whole slope to have a major influence on stability. The phreatic surface was therefore gradually increased in three increments, 0.1 m, 0.25 m and 0.5 m above measured maximum values. Each increment shows a distinct decrease in the overall Factor of Safety of this site. For example, assuming a residual friction angle of approximately $20^{o}$, the Factor of Safety drops to 0.77, 0.49 and 0.42 respectively. These specific changes are reflected in the overall matrix of values. For example, with large rises in the phreatic surface, stability requires high residual cohesion and friction parameters not likely to be experienced within these materials. It is clear therefore that phreatic surfaces much higher than those measured in the field are unlikely to exist and that failure will occur long before these conditions are likely to be reached. It can therefore be assumed that the measured field conditions used in this analysis are close to the maximum values likely at this location. This is also reflected in the results of the geotechnical analysis (chapter IV) and field monitoring program (chapter V). Finally, stability analysis was conducted with decreased pore water pressure values. The phreatic surface was lowered by 0.1 m and 0.25 m. As would be expected, the Factor of Safety gradually increases and, at the relevant residual friction angle for this site, the slope is almost stable with a 0.1 m fall in head with a Factor of Safety of 0.99. It appears therefore that, assuming other parameters are constant, this phreatic surface almost represents critical conditions between those of stability and instability. This is reflected across the Factor of Safety matrix. With a further drop in pore water pressure, to a level 0.25 m below measured maximum field conditions, the slope is stable with a Factor of Safety of 1.16 at zero residual cohesion and residual friction angle of $20^{o}$. It therefore appears that for a pore water decrease of 0.1 m the slope is only just unstable and that at a reduction of somewhere between 0.1-0.2 m head of water stability results. Field monitoring (chapter V) confirms this and suggests that at measured maximum pore water pressure values, the phreatic surface is high enough to produce surges in movement. At lower phreatic values movement will continue but at slower rates, while it will cease with a drop in pore water pressure in excess of 0.1 m. It seems therefore that the slope is sensitive to changes in the phreatic surface and that failure conditions can be established with reasonable accuracy, these confirming the frequency and seasonality of landslipping monitored in the field. Also, if the variability of precipitation and other controlling parameters are borne in mind, it appears that this analysis might well explain the observed phenomena of episodic movement known as 'stick-slip' (Brunsden, 1984).

6.3.2.3 Mupe Bay

The results for Mupe Bay are presented in Table 6.3. Initially some difficulty was experienced, with the computer program refusing to accept some of the slope geometry details. Modification of the input was therefore required. This can be achieved without significantly altering the outcome of the analysis (Cobb, pers. comm.). A number of tests were conducted in the first instance to establish the most suitable slope geometry. This highlights one of the advantages of such a flexible program, namely, the manipulation of variables in an attempt to elucidate unknown parameters and highlight potential sources of error. Stability analyses results suggest that at this site the program tends to produce low Factors of Safety. With measured field parameters the results suggest a highly unstable slope. For example, for the laboratory determined residual friction angle the Factor of Safety is 0.64-0.70, assuming zero residual cohesion. In fact with no cohesion, a friction angle of between 36-38$^{o}$ is required, to give a Factor of Safety of unity. The level of the phreatic surface was lowered, firstly by 0.1 m and then by 0.25 m. In both cases at zero cohesion and residual friction angle of 27-28$^{o}$ the slope remains unstable with Factors of Safety of 0.73-0.79 and 0.84-0.92 respectively. An element of residual cohesion is required for the slope to be stable at these values. If previous work by Craig (1981) is considered, this may account for the low Factors of Safety produced by the stability analyses. This cohesive influence is particularly likely at this site, where the slide is shallow, all piezometers were empty for long periods over the summer and pore water suction may be assumed to be significant. However, although this may be important and that as a consequence the results of the Stability Analysis are more plausible, further work is required to verify the likely effects of cohesion. Finally, the level of the phreatic surface was raised, by 0.1 m and 0.25 m. As would be expected, Factors of Safety drop significantly in both instances. Clearly such pore water pressures would not exist in reality since failure would result before such increases could be reached. For example, at zero residual cohesion and a residual friction angle of $27^{o}$ the Factor of Safety is approximately 0.55-0.60 for a 0.1 m drop in head and 0.42-0.46 for a 0.25 m drop in head. Field conditions (Chapter V) confirm this. For example, in wet months the steeper slope sections are bare; material is unable to remain <u>in situ</u> on the surface and the slip surface is exposed.

The Factor of Safety matrices all therefore suggest that the slopes at Mupe are currently particularly unstable. However, it must be remembered that the initial input had to be modified and therefore the results must be regarded with some caution.

6.3.2.4 Worbarrow Bay

Input was acceptable for analysis, indicating that measured geometries and material properties are likely to be representative of site conditions. Results are presented in Table 6.4. Stability analysis was conducted to represent measured material and slope properties, summer phreatic conditions by calculating Factors of Safety with maximum toe slope pore water pressure but no water in the remaining piezometers, overall increased phreatic surface values and finally a lowered phreatic surface.

At the measured maximum phreatic surface, results suggest that the slope is unstable. At zero residual cohesion and a residual friction angle of 26-27$^{o}$ the resulting Factor of Safety is 0.42-0.5. There is some indication therefore that the stability analysis has underestimated the Factor of Safety. Indeed, at a residual friction angle of 26-27$^{o}$ residual cohesion has to be at least 2.5 $kNm^{-2}$ for a Factor of Safety of unity. However, it should be noted that the mean slope angle measured in the field is 21$^{o}$. This was obtained immediately following a period of extensive movement (see chapter V) and presumably resulted from slope conditions surpassing the critical angle. From these results it appears that the measured pore water pressure values are likely to be close to the maxima possible on this slope. There seems little point therefore in conducting extensive stability analysis at higher pore water pressures. One analysis only was therefore conducted, assuming an overall increase in the phreatic surface of 0.1 m. As expected, the Factor of Safety drops to very low levels. For example, assuming zero cohesion and a residual friction angle of 26-27$^{o}$ the Factor of Safety drops from 0.42-0.50 to 0.33-0.39. These conditions would clearly not occur in reality. However, results do confirm that landslipping will occur every season as the phreatic surface rises. Stability analysis was further conducted, with input values representing a drop in the level of the phreatic surface, of 0.1 m and 0.25 m. At both levels, the Factor of Safety remains below unity for measured site cohesion and friction angle characteristics. A number of reasons can be suggested for this. Residual cohesion is important. The Factor of Safety matrix indicates that at small cohesions, stability increases significantly to values of greater than 1.0. Secondly, the measured phreatic surface is particularly high relative to mean conditions throughout the year and also measured residual shear strength is possibly too low. This is likely if, for example, there are major undulations across the shear plane or if cohesion (Craig, 1981) is important. Finally, some variables which are significant in controlling the overall slope stability may not be included in the analysis.

Finally, analysis was conducted in an attempt to represent summer conditions (see chapter V), with a high toe slope phreatic surface but zero pore water pressure along the remainder of the slide. In these circumstances the Factor of Safety matrix suggests that at zero residual cohesion, and friction angles of 26-27$^{o}$, the Factor of Safety is just greater than unity. However field conditions, including likely cohesion in summer months and pore water suction towards the rear of the slope, are likely to result in slightly higher Factors of Safety than the stability analysis suggests. With high pore water pressures in the toe region this section of the slope is likely to be unstable for much of the year. However, it must be remembered that the leading edge of the mudslide is at a low angle and passes out over shingle. Consequently, water flowing out of the slip surface at the toe of the slope will drain away. The leading edge of the mudslide will therefore be stable, supporting the area of high pore water pressures immediately behind it. Thus the effects of high pore water pressures at the toe only seem likely to be important with a high phreatic surface throughout the remainder of the slide.

### 6.3.2.5 Conclusions

Results from all four sites permit some general conclusions to be drawn. By presenting Factor of Safety matrices, more instructive use can be made of previously measured parameters and much detail can be thrown on the stability characteristics of the studied slopes. Conducting analysis in this way also permits conclusions to be drawn through both time and space as to the characteristics of the instability. The stability of each slope depends on a variety of parameters. By assuming these remain constant, the effects of seasonal changes in the phreatic surface can be considered. Where the cliffs appear to be quasi-stable, the Factors of Safety are close to unity and the slopes appear to be sensitive to changes in the level of the phreatic surface. It should also be noted that in some circumstances results suggest that Factors of Safety have been overestimated. This is likely to be a result of localised field conditions and the fact that analyses such as these are frequently designed to identify the worst likely site conditions (Cobb, pers. comm.). Finally, results confirm the seasonal frequency of landslipping and the nature of 'stick-slip' movements during the winter, although further work is clearly required on the importance of 'cohesion' at residual strength and the possible effects of pore water suction during dry seasons.

It can thus be concluded that stability analysis has provided useful additional details to this work and has shed much light on the effects of measured material properties, mechanisms of failure and previous interpretations.

## 6.4 STABILITY ANALYSIS OF PORTLAND LIMESTONE

The geotechnical study of the Portland Limestone (chapter IV) provides the base for discussing the results of the stability analysis. Since many of the parameters used were derived from the laboratory test program this investigation must follow two courses. Firstly, an investigation of discontinuity orientation. This is particularly relevant to the Portland Limestone because it is the major determinant of failure type. Relationships between bedding and joints are often critical to overall cliff development, and the orientation of discontinuities relative to the cliff face is important to the examination of slope stability relative to failure type (Hoek & Bray, 1981). Secondly, the application of stability analysis models is a natural progression from previous field (chapter III) and laboratory (chapter IV) investigations.

### 6.4.1 Discontinuity Analysis

The orientation of discontinuities was recorded along the Portland Limestone outcrop and sampling conducted at those points identified and examined as part of the geotechnical investigation. Consequently, all Portland Limestone coastal outcrops were studied and measurements were also taken at one point where the unit swings inland between Gad Cliff and St Alban's Head. Data were collected using previously discussed techniques but, due to the employed analysis technique and required statistical accuracy of the results, additional procedure was adopted. Data collected for the joint rosettes (chapter IV) were used as the base of a more detailed and larger sample. Many studies have been conducted to investigate sample bias and size requirements, to ensure statistical accuracy of interpreted results (Hoek & Bray, 1981; Hudson & Priest, 1983; Priest & Hudson, 1981; Terzaghi, 1965). Particular care was therefore taken to collect an adequately large number of readings, since critical discontinuity patterns can only be identified if sufficient information is present. To achieve this each surface was measured twice and on those occasions where a large difference was present between the two readings, a third was recorded. These were averaged to give the result. Consequently, approximately 150 readings were taken at each site but complexities due to large numbers of individual readings, particularly following plotting on interpretive diagrams, were reduced. This procedure also minimised operator error and the influence of inaccurate readings. Approximately the same number of measurements were taken for each identified discontinuity set. Fractures not appearing to belong to a specific group were also recorded.

Data is represented using hemispherical projections. A comprehensive discussion of the specific methods used in this study is widely reported in available literature (Badgley, 1939; Hobbs et al, 1976; Phillips, 1979; Priest, 1985; Ragan, 1973). Beavis (1985) regards such techniques as "essential to the geomechanical study of a rock mass, permitting the definition of sets and systems (of discontinuities) and forming the basis for kinematic analysis", while Hoek & Bray (1981) suggest that it is particularly useful to a study of this type because "data is presented in such a way that it can be easily evaluated and incorporated into stability analysis". Equal area projections were adopted for this study. Results are presented in two forms; equatorial equal-area stereonets, marked at $2^{o}$ intervals, with data representation on the lower hemisphere and contoured polar equal-area stereonets, marked at $2^o$ intervals, with data representation on the lower hemisphere. The plotting of both lines representing planes and points representing poles was conducted using the guidelines suggested by Hoek & Bray (1981). The contouring of the poles was undertaken following the technique discussed by Priest (1985). The results are discussed in three parts; the identification of sets of discontinuities, aspects of the discontinuity sets identified from the stereonets which are important to this study and the confirmation of mechanisms of failure evident from the data and their spatial distribution.

#### 6.4.1.1 Identification of Discontinuity Sets

A number of sets of discontinuities can be identified from the stereonets. Bedding; dips towards the north but at varying angles between horizontal in the east to almost vertical at the western end of the outcrop. Discontinuity set 'A'; joints striking north-south with particularly steep angles of dip, some plunging to the east and others to the west. Discontinuity set 'B'; joints striking east-west, dipping towards the south but with increasing angles of dip towards the east, where the discontinuities are almost vertical. Discontinuity set 'C'; joints striking north-east/south-west and dipping towards the south east. Discontinuity set 'D'; joints striking north-west/south-east which dip towards the south-west. Discontinuity set 'E'; other discontinuities which do not appear to be associated with any of the dominant groups. Many of these appear to be randomly oriented while others may represent fractures associated with one of the above sets but which, for some reason, display a greater variability than their partners.

The joint rosettes (chapter IV) suggest patterns such as these may be present but the stereographic plots provide much clearer evidence of this and complement the geotechnical survey.

6.4.1.2 Detailed Characteristics of the Discontinuity Sets

Conclusions can be drawn from the fracture patterns regarding their relationship with structure (see chapter II) and slope stability. The importance of structure and the changes it imposes on the stability of the coastal cliffs can be firstly examined by considering bedding. This reflects the oblique nature of the Purbeck monocline relative to the coast.

In the east, bedding is horizontal (Figures 6.5-6.7). Consequently, on the equatorial nets, bedding plots as a circle around the edge of the diagram, while on the polar nets bedding is absent. It does in fact show up at Tillywhim (Figure 6.5) but this is only due to undulations across one or two of the surfaces, as indicated by the low density contour. This is likely to be due to the extensive faulting along the coast in this area (Arkell, 1947). At Emmett's Hill (Figure 6.7) poles contoured at the centre of the net, in a tight cluster, indicate slight northerly dip of almost uniform direction. The changing dip recorded at Emmett's Hill, a consequence of the oblique nature of the monocline, is reflected in the remaining plots (Figures 6.9-6.16), with increasing angles of dip represented by the movement of the poles representing bedding towards the edge of the net. Contouring remains close, however, indicating uniformity in dip direction within each site. Although the contoured poles indicate high point densities, representing uniformity between measurements, lower levels of contour do suggest slight

Key for Figures 6.5 - 6.16

Percentage values per 1% area

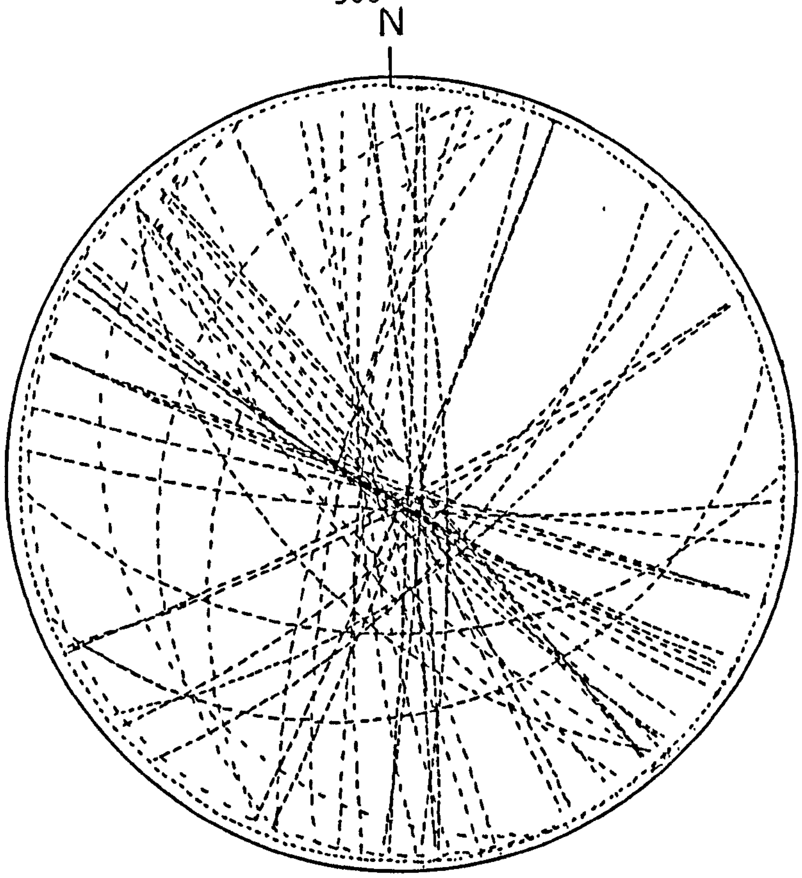
0 to < 2%

>= 2% < 4%

>= 4% < 6%

>= 6% < 8%

>= 8%



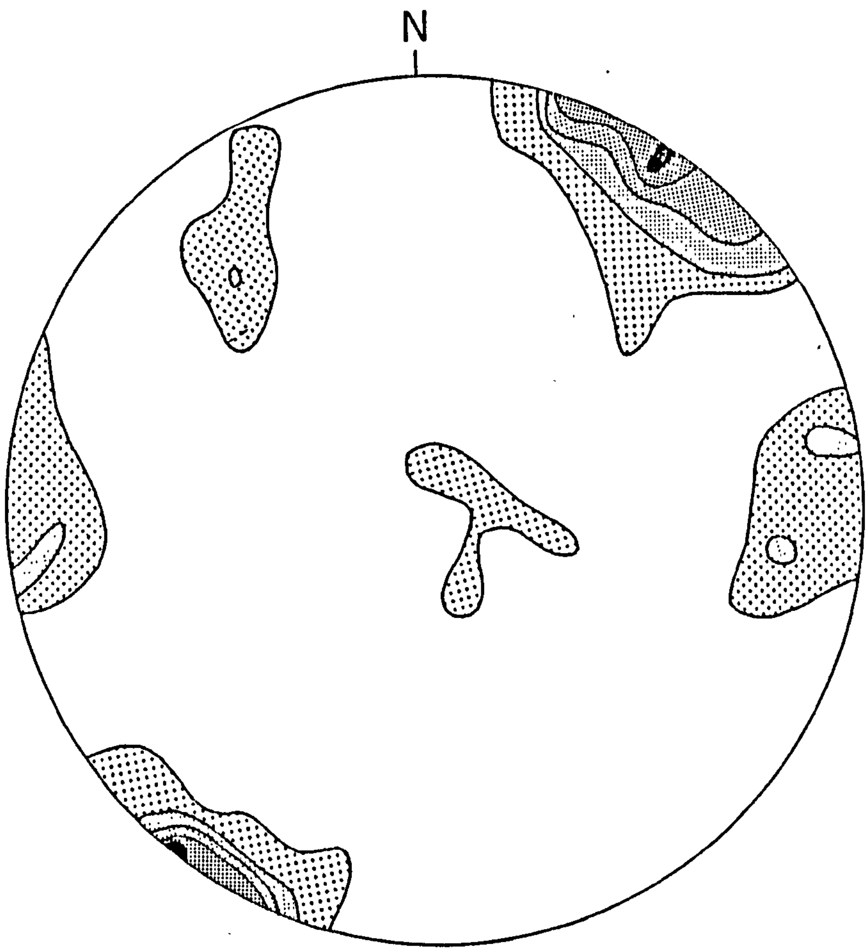




FIGURE 6.5

Equatorial and polar projections for discontinuities in Portland Limestone at Tillywhim

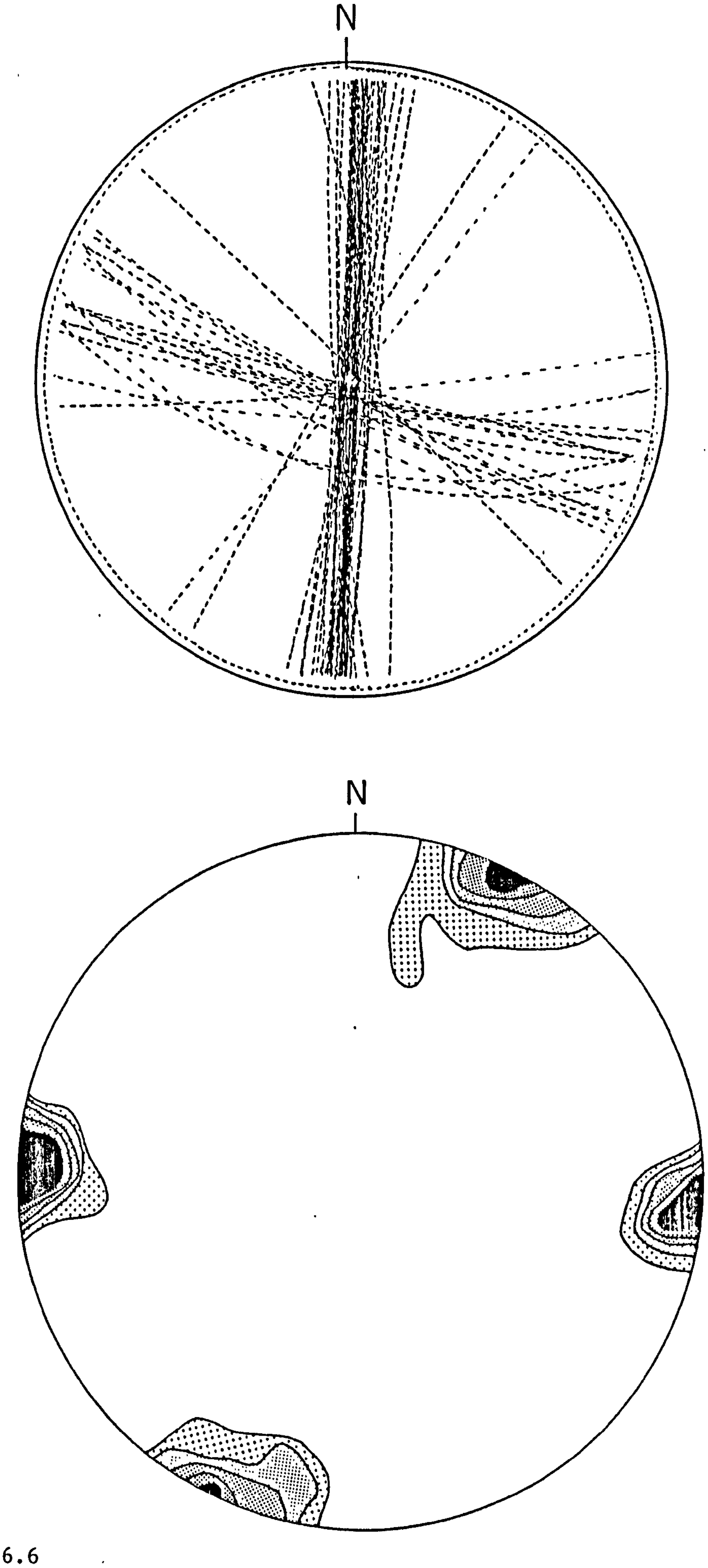
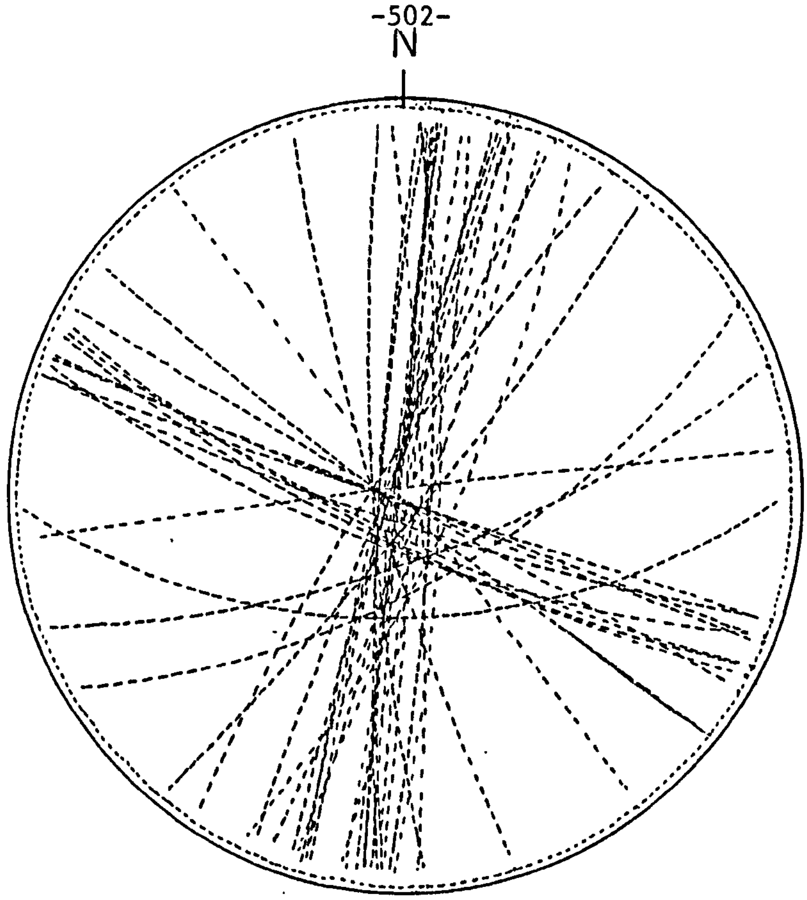



FIGURE 6.6

Equatorial and Polar projections for discontinuities in Portland Limestone at Seacombe



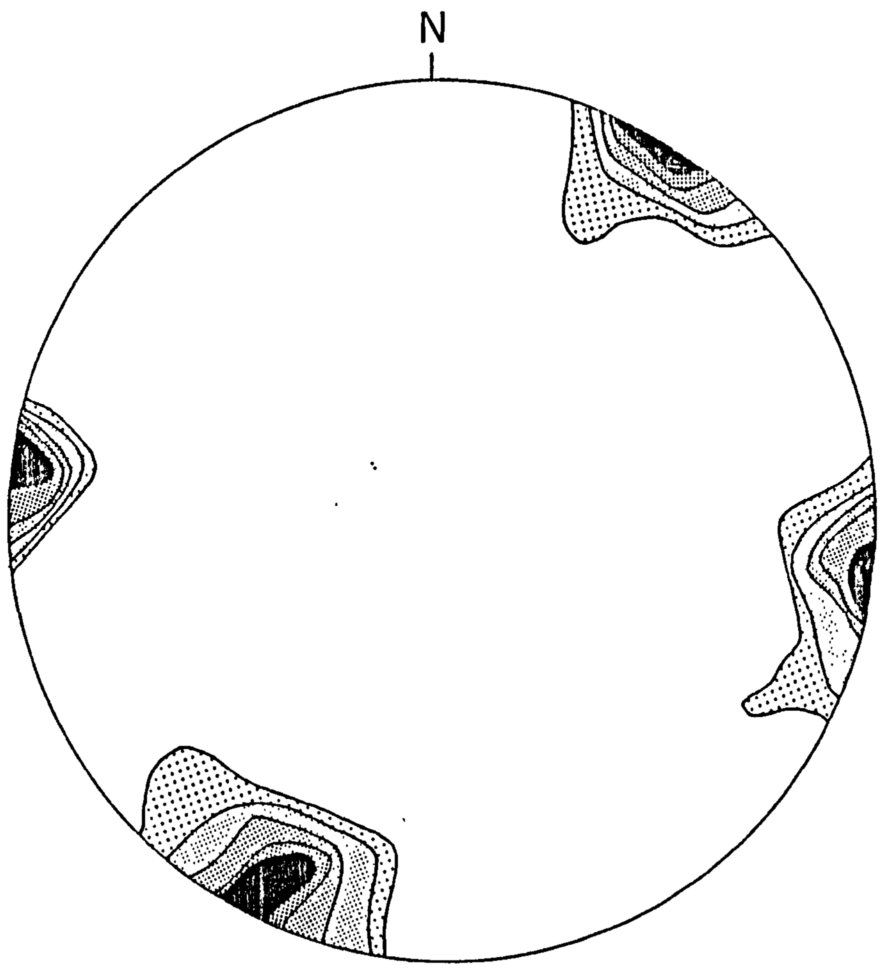




FIGURE6.7

Equatorial and Polar projections for discontinuities in Portland Limestone at Winspit

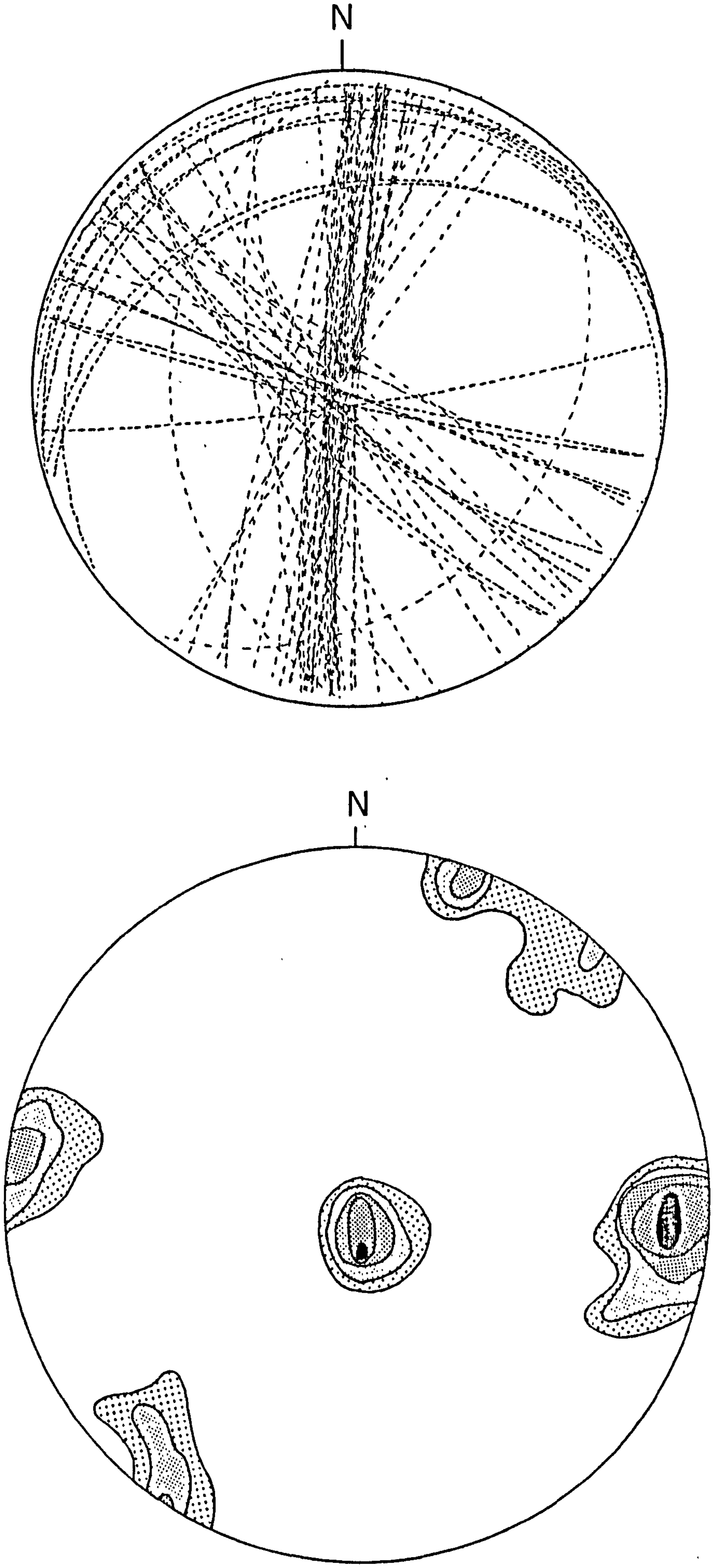



FIGURE 6.8

Equatorial and Polar projections for discontinuities in Portland Limestone at Emmetts Hill

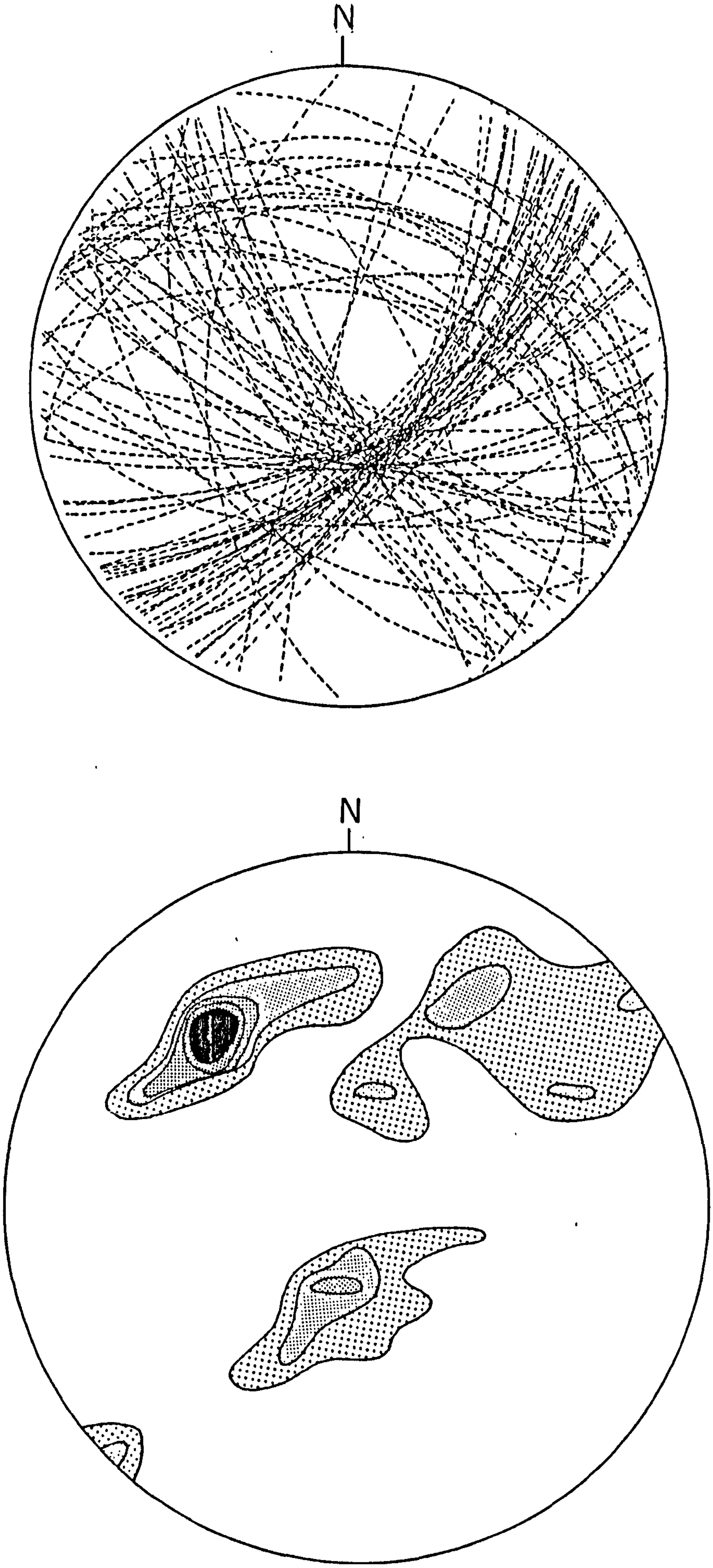



FIGURE 6.9

Equatorial and Polar projections for discontinuities in Portland Limestone at Kimmeridge

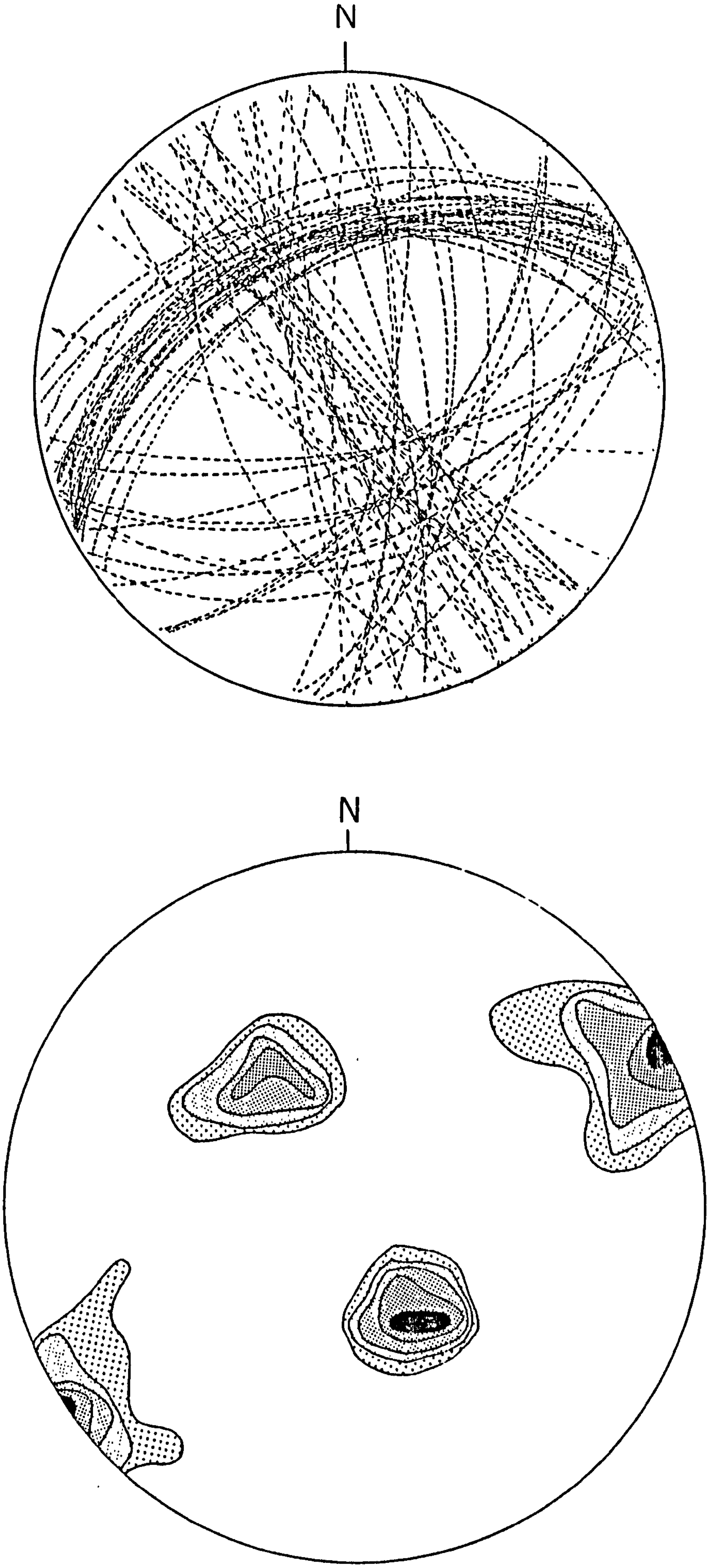



FIGURE 6.10

Equatorial and Polar projections for discontinuities in Portland Limestone at Pondfield

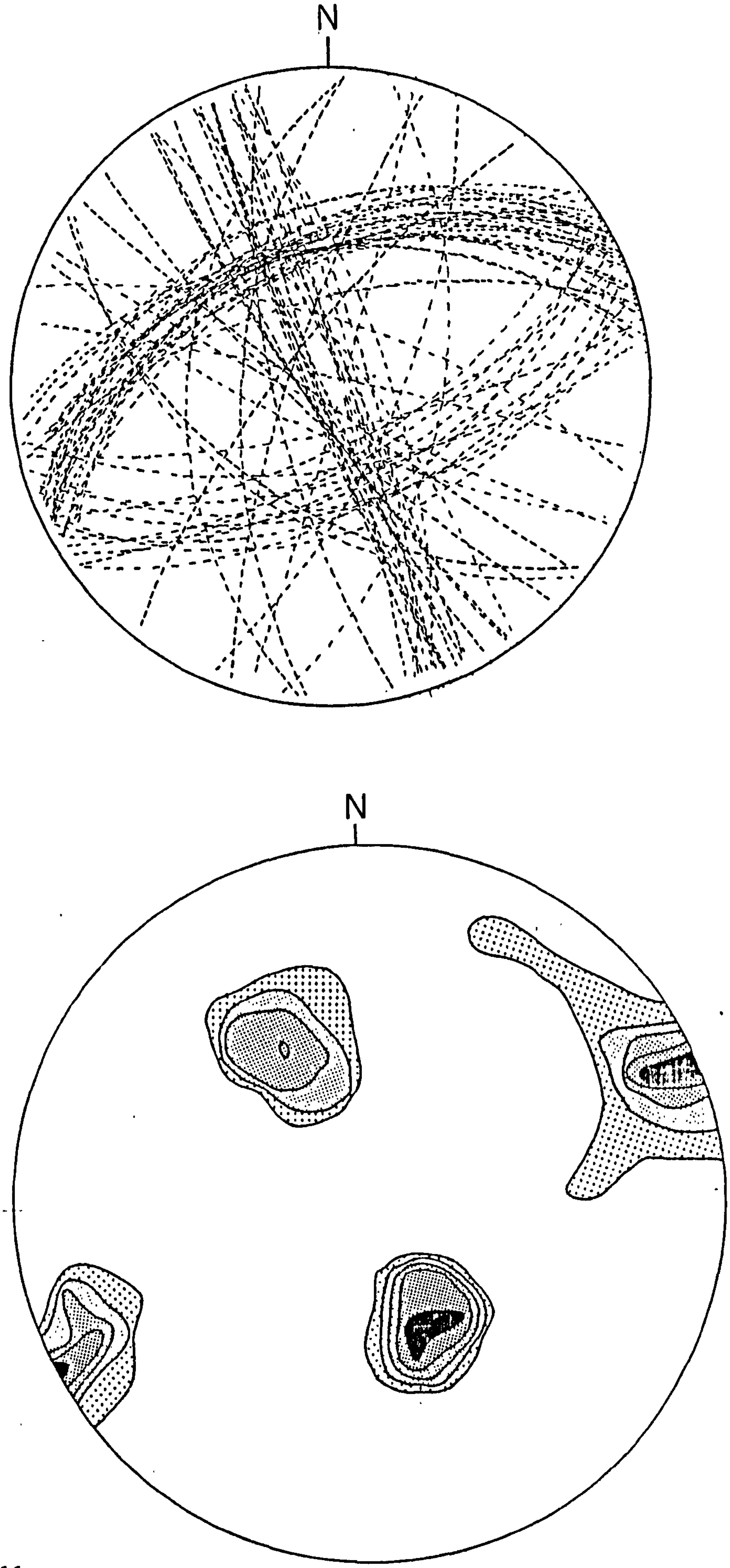
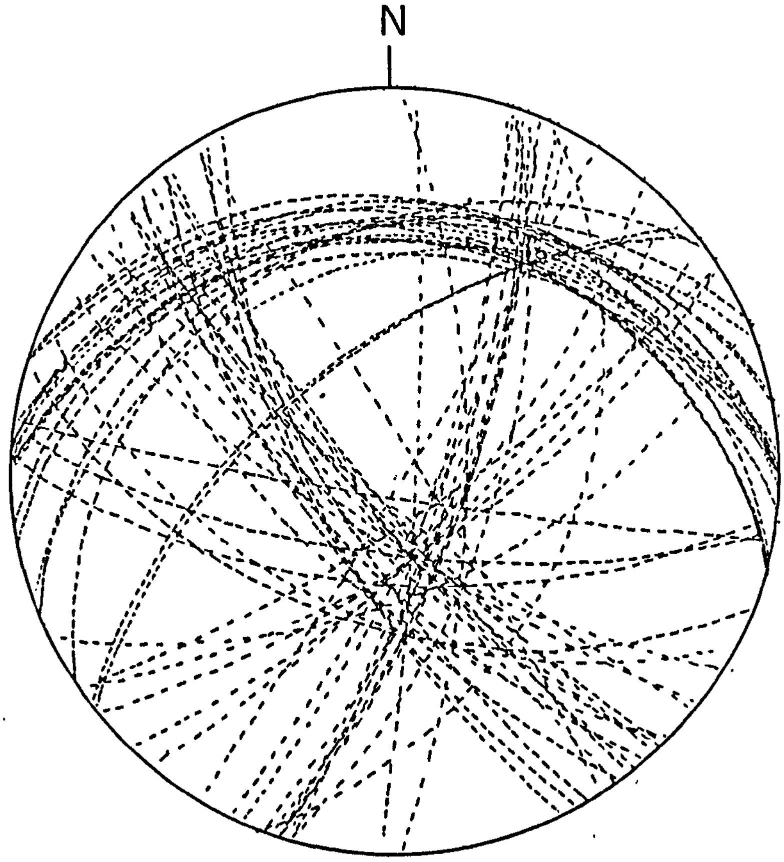



FIGURE 6.11

Equatorial and Polar projections for discontinuities in Portland Limestone at Worbarrow Tout



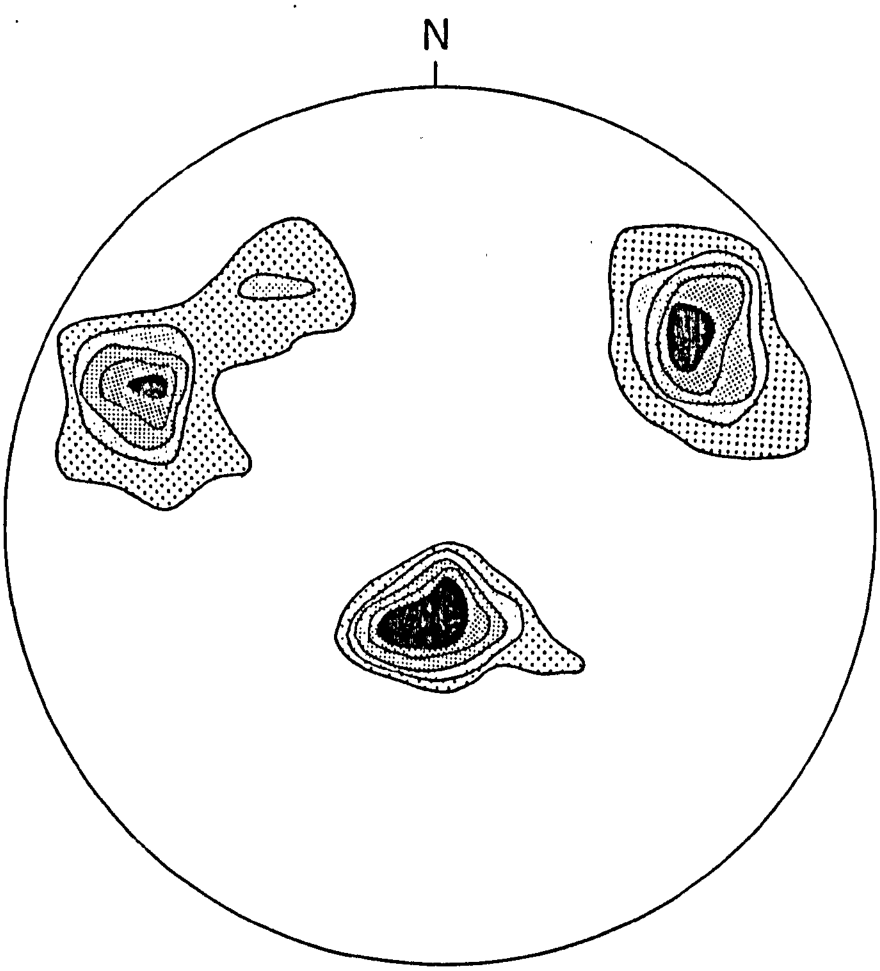




FIGURE 6.12

Equatorial and Polar projections for discontinuities in Portland Limestone at Bacon Hole

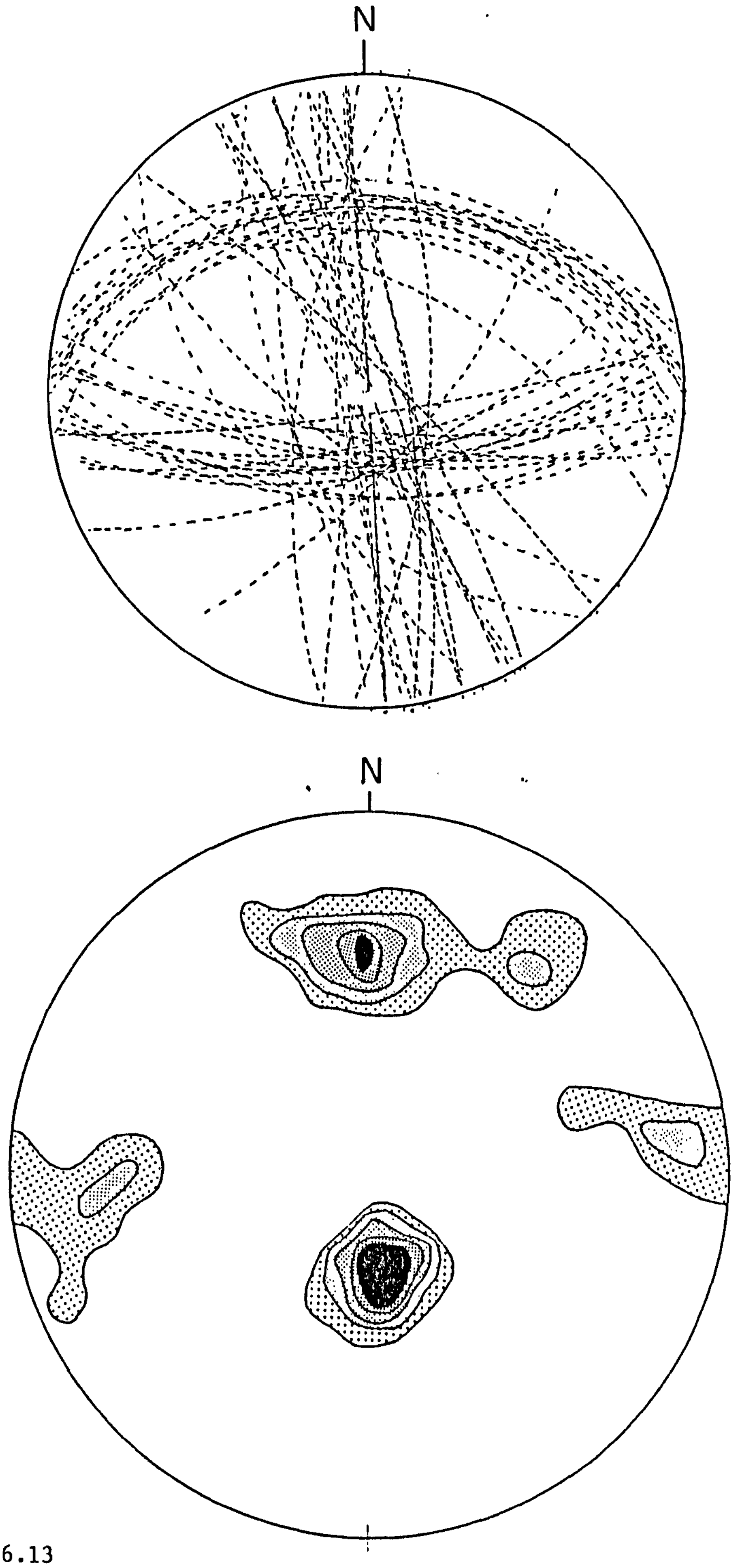
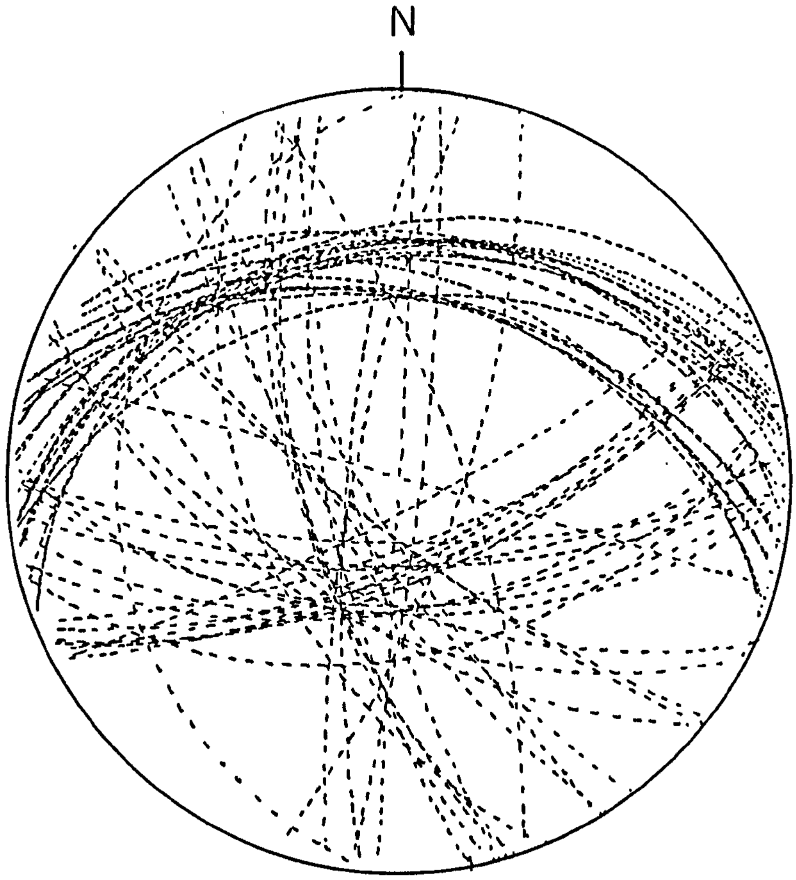



FIGURE 6.13

Equatorial and Polar projections for discontinuities in Portland Limestone at Fossil Forest



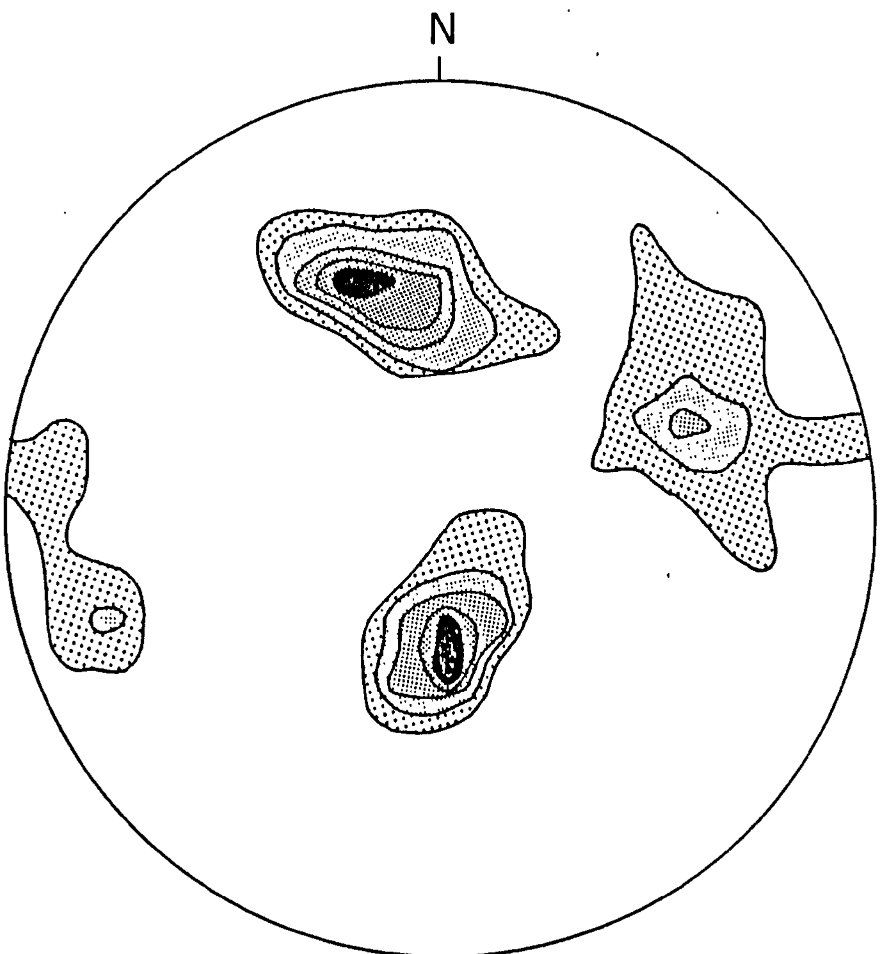
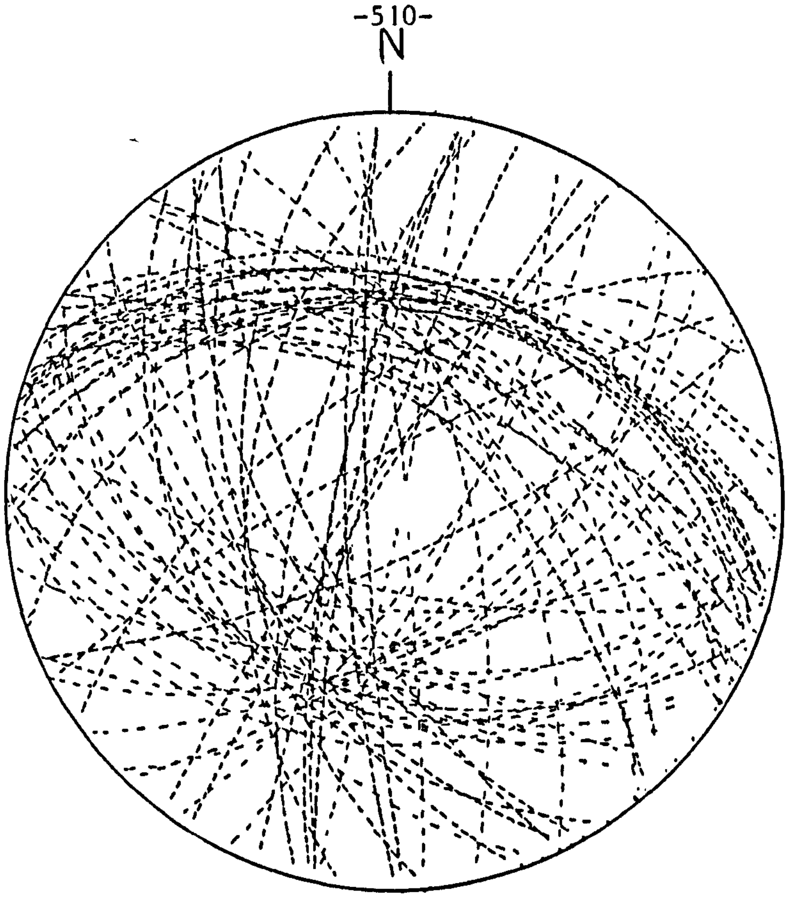




FIGURE 6.14

Equatorial and Polar projections for discontinuities in Portland Limestone at Lulworth Cove



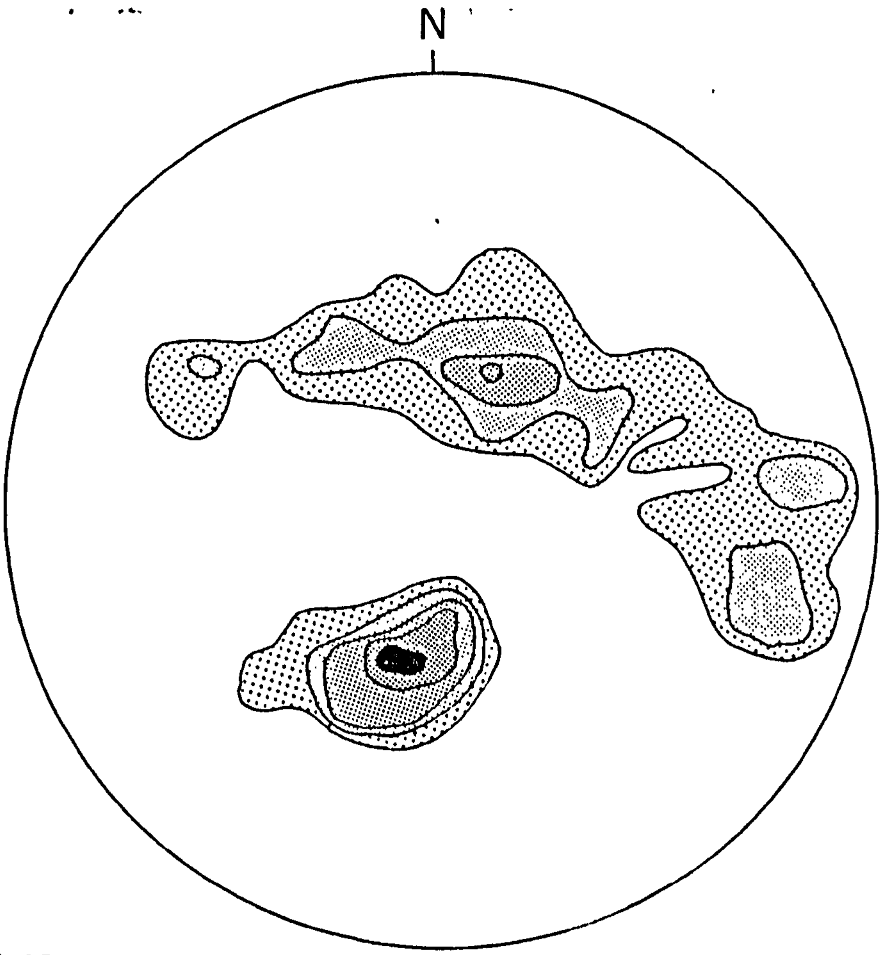
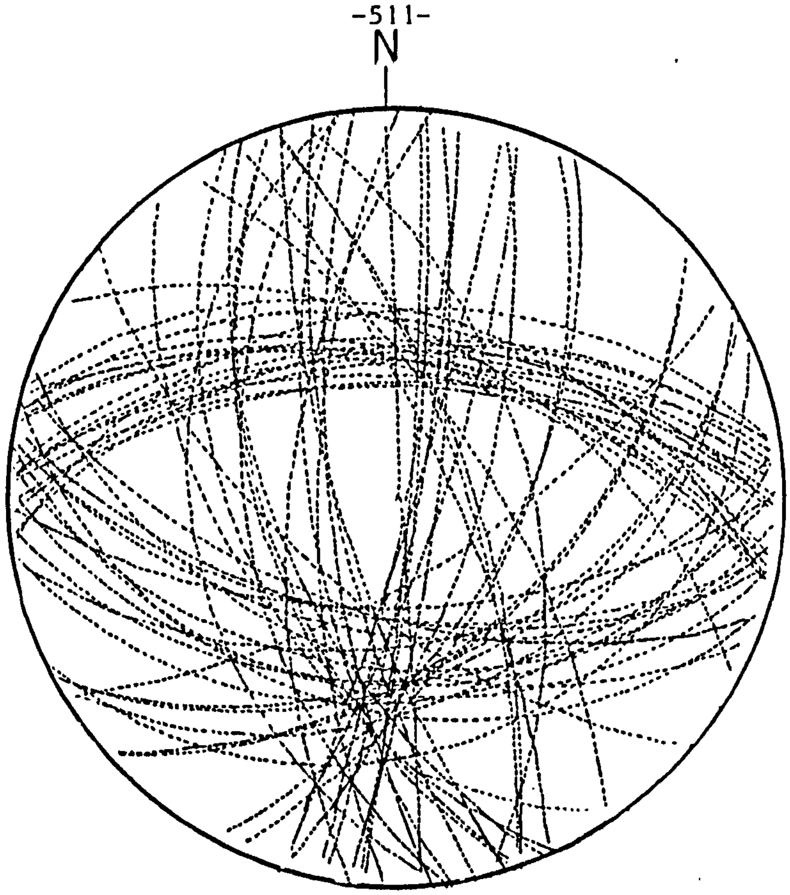




FIGURE 6.15

Equatorial and Polar projections for discontinuities in Portland Limestone at Stair Hole



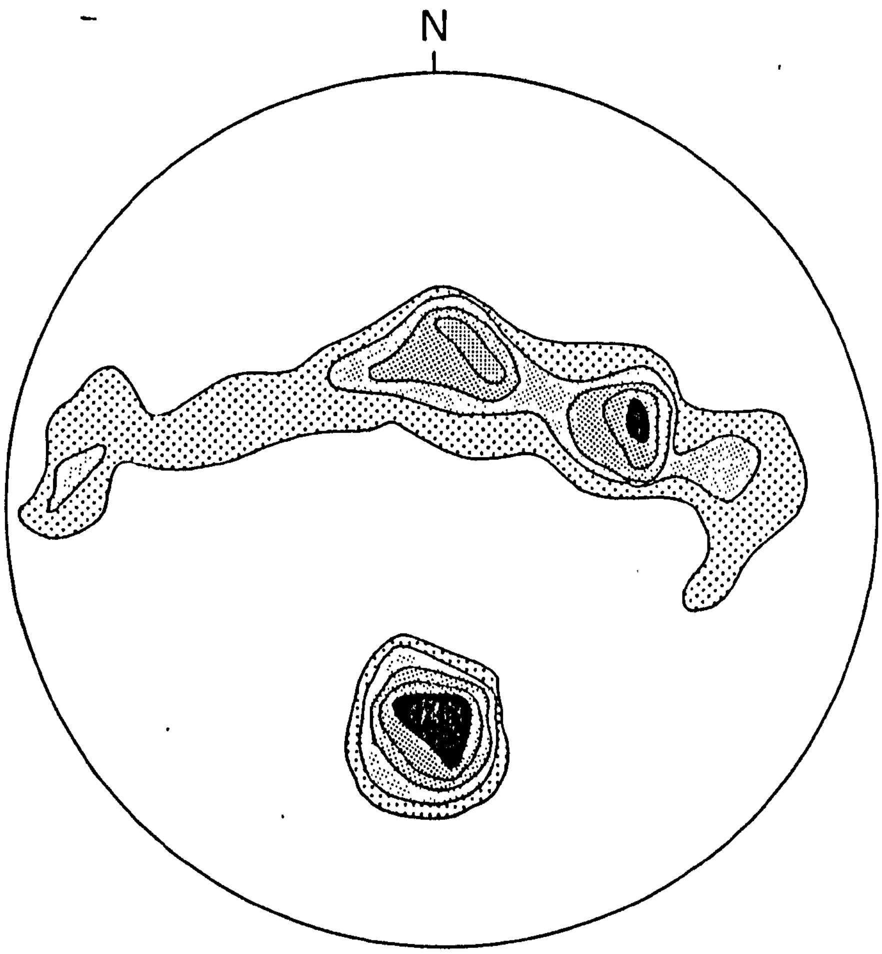




FIGURE 6.16

Equatorial and Polar projections for discontinuities in Portland Limestone at Durdle Door variation in direction. For example, at Worbarrow Bay (Figure 6.11) and Pondfield (Figure 6.10) bedding assumes a slight north-westerly attitude, while at Stair Hole (Figure 6.15) this has shifted towards the north-east. These trends to bedding are significant, not only because they form a continuum upon which the remaining results can be discussed, but also because such major changes are bound to have significant effects on mechanisms of failure and cliff stability.

By further considering the identified joint sets in relation to bedding, the following spatial site groupings can be identified:

Group A. This includes the cliffs at Tillywhim (Figure 6.5), Seacombe (Figure 6.6), Winspit (Figure 6.7) and Emmett's Hill (Figure 6.8) where bedding is almost horizontal and indeed completely flat at two locations. Joint set 'A' is present at all sites as is joint set 'B'. These can all therefore be considered characteristics of that part of the Portland Limestone outcrop which occurs in the flat lying section of the fold structure.

Group B. This includes all other coastal cliffs (Figure 6.10-6.16) and the inland section of the outcrop (Figure 6.11). These sites have been grouped based on four characteristics. Bedding dips to the north, joint set 'A' is present, despite some variation in the precise direction and angle of dip, joint set 'C' is visible at all sites and joint set 'D' is also present. These characteristics are clearly seen on both equatorial and polar stereonets. However, a more detailed examination of the discontinuity characteristics suggests two sub-groups exist within this main division.

Group B(i) includes the coastal cliffs around Pondfield (Figure 6.10), Worbarrow Tout (Figure 6.11), Bacon Hole (Figure 6.12), Fossil Forest (Figure 6.13), Lulworth (Figure 6.14), and the inland measurements at Kimmeridge (Figure 6.9).

Group B(ii) includes the outcrops at Stair Hole (Figure 6.15) and Durdle Door (Figure 6.16). These sub-divisions are based on a number of observations. Contouring of the poles indicates a high degree of regularity to both the angle and direction of dip of joint sets in group B(i) which is absent in group B(ii). This is seen in the increased complexity of the equatorial nets at Stair Hole (Figure 6.15) and Durdle Door (Figure 6.16), for example, and the correspondingly low density contour plots at these two sites. At the lower contour intervals at Stair Hole and Durdle Door, set 'A', 'C' and 'D' joints do not form distinctly separate groups as they do at the other sites. Also group E joints appear to have greater predominance in group B(ii) than they do in B(i) sites. Two hypotheses can be offered in an attempt to explain these variations and thus the structural influence on mechanisms of failure. The transition from joint set 'A' to groups 'C' and 'D' as the dip of the bedding gradually increases to the north, simply represents the separation of two discontinuity sets. Where bedding is flat, all joints are particularly steep and it is therefore difficult to isolate individual sets of discontinuities striking in approximately the same direction. Associated with increased angles of dip to bedding is a reduction in the angle of dip of joints and a slight swing to the strike, making it possible to separate the measurements represented by set 'A' into its component parts. This change depends on the proximity of the coastal cliff to the steep limb of the fold, which causes increasing complexities in the discontinuity patterns as the rocks become more highly fractured, and a consequent increase in the number of group 'E' discontinuities at Stair Hole and Durdle Door. A second hypothesis is that there is no relationship between group 'A' and group 'C' and 'D' discontinuities. Set 'A' fractures represent the results of compression in the central limb of the fold, while groups 'C' and 'D' are conjugate discontinuities present in the steeply dipping section of the structure and are indicative of the complexities not only present in the Portland Limestone but also other materials, such as the Chalk, where bedding is particularly steep.

From these results it is impossible to definitively conclude which of the above alternatives is correct. Further work is required. This would require the sinking of boreholes along the inland outcrop where exposures are not present and would thus form a useful extension to these results. Despite these problems, however, the above discussion permits the clear identification of failure mechanisms and their spatial distribution.

### 6.4.13 Mechanisms of Failure

The mechanisms of failure proposed in Chapter III can now be examined in some detail. The polar stereonets at Tillywhim (Figure 6.5), Seacombe (Figure 6.6), Winspit (Figure 6.7) and Emmett's Hill (Figure 6.8) indicate rectangular blocks of Portland Limestone are present due to the bedding and joint sets A and B. The size of each will depend on fracture density but clearly, detachment of material will be in the form of individual blocks, with occasionally more than one becoming dislodged at the same time. Field evidence supports this. The joint pattern produces cuboidal blocks (Plate 6.1). At some locations, major overhangs occur along the cliff (Plate 6.2) indicative of the stability of the cliffs in this region. Numerous caves and quarries, a consequence of building stone extraction, provide further evidence of these forms. Their presence alone highlights the stable state of the in-situ material. Occasionally, individual blocks have fallen out of a cave roof but the surrounding material has remained in-situ. Similarly, buckling of the roof has occurred in many instances but without failure, providing further evidence of both the detachment mechanism and the stability of the cliffs.

At the remaining sites the polar stereonets are characteristic of wedge failures, similar to those described by Hoek et al. (1973). These are directly controlled by joint sets C and D but from the foregoing discussion it will be clear that bedding and joint set A will also exert some influence on the precise nature of the failure. For example, changes in the relative geometry of each set of discontinuities will alter the size of each wedge and ease of detachment.

Previous discussion (chapter III) suggested that cliff collapse at Durdle Door occurs by toppling alone. With these results this assumption can no longer be upheld. Contoured polar stereonets now suggest that wedge failures are also present at Durdle Door and are responsible for the detachment of significant amounts of material. Toppling is likely to occur, due to the orientation of bedding relative to the angle of the cliff face, but it cannot be assumed to be as important as was previously suggested. Hoek & Bray (1981) recognise four types of topple. Flexural, where continuous columns of rock are separated by steeply dipping discontinuities and breaks of flexure occur due to toe

PLATE 6.1

Joint pattern in the Portland Limestone at Winspit

PLATE 6.2

Overhang in the Portland Limestone at Seacombe

slope undercutting; block, due to individual columns being separated by widely spaced orthogonal joints; block-flexure, a result of accumulated flexure along cross-joints and secondary topples, where the initiating process is external to the cliff itself. It appears that block toppling may occur at Durdle Door but this cannot be made as a definitive statement since, due to restricted access, it has been impossible to take detailed measurements at sea-level along this cliff section. Such situations, where two failure mechanisms can occur together, have been recorded at other sites. Hoek & Londe (1974), for example, identify sites where combinations of discontinuities lead to toppling as well as wedge failures.

Finally, these results can also be used to present preliminary suggestions on the occurrence of natural arches at Durdle Door and Stair Hole. At Durdle Door the association of near vertical bedding and joint sets 'C' and 'D' produce wedges approximating in their orientation to a 'V'. Joint bounded blocks lying around an opening in this material will therefore be inherently stable, assuming the geometry of a parabola, interlocking and relying on each other for support with loading transferred to the supporting columns without the question of lateral stress. The dislodgement of one block will not, therefore, result in the collapse of the whole structure but only the removal of other adjacent blocks relying on the first for support. It is the discontinuity pattern which accounts for the development of such a feature at Durdle Door and why similar features do not occur elsewhere along the coast.

Two arches are also present at Stair Hole, but their morphology differs from that at Durdle Door. This can again, however, be explained by recourse to the stereonets and the differences in the discontinuity pattern between the two sites. At Stair Hole the dip of bedding is less steep. This explains the extended width of the outcrop, within which the arches have formed. Material removal is predominantly by wedge failure and, due to the angle of dip of bedding, the 'keystone' principle seen at Durdle Door seems to be less important, with joint friction appearing to play a more important role. The opening out of each arch has been due to the intersection of steeply dipping bedding planes and joint set 'A', these produce blocks of material whose geometry is that of an inverted 'V' and is therefore inherently unstable. Those blocks which remain form little more than a remaining shell to the original outcrop. They are gradually becoming dislodged, sliding out and leaving spaces which subsequently widen. It is predicted that continued material removal along these lines will eventually result in a complete breach similar to that already present at Stair Hole, to the west of the two arches. Despite the stereonets providing much detail regarding the current morphology of these features, the reasons behind their initiation remain unclear. Although suggestions can be made, including localised structural irregularities, or cliff sections being particularly susceptible to marine attack, it is not currently possible to identify the cause behind the initial development of these openings.

6.4.2 Determination of Factors of Safety

The detailed consideration of discontinuity patterns and associated mechanisms of failure, goes some considerable way towards explaining variations in the stability of the Portland Limestone cliffs. However, the foregoing discussion dwells only on the orientation of discontinuities. While this is clearly a dominant control, the overall stability of the cliff will also depend on other parameters. As previously noted, slope stability analysis techniques for 'hard' rock slopes are rare. However, a computer based model which can be applied to free faces was identified, which appeared to be relevant in this instance. The 'Tetrahedral Block Analysis' program used here, as with the stability analysis program used for the Wealden Beds, is utilised commercially by engineering geologists (Geoffrey Walton, Consultant Engineering Geologists, pers comm.). Consequently it is not possible to give details of the analysis routines although the data required for the analysis and input format can be presented. Despite this drawback the discontinuity patterns identified from the stereonets suggest that Tetrahedral Block Analysis is particularly suitable here (Cobb, pers. comm.), hence the decision to utilise the technique.

### 6.4.2.1 Data Requirements: Input Format and Sampling

The data required in these analyses include mean dip of bedding using measurements made with a compass clinometer; cliff free face angle, using data collected with an Abney level, siting from the toe of the cliff to the crest; cliff height, calculated from two aneroid barometers following the technique described by Pugh (1975); angle of dip of the basal block in the cliff section (which in this instance is also the bedding plane), measured with a compass clinometer and the length of the daylighting face of the first and second blocks in the cliff profile, measured using a linen tape. Readings were taken at each coastal site sampled as part of the geotechnical investigation. Two sites were excluded. Emmett's Hill was omitted because the Limestone exposure forms only part of a composite cliff of Portland Stone, Sand

and Kimmeridge Clay. To present Factors of Safety based on the one rock unit would therefore be grossly misleading. Kimmeridge Quarry was not utilised because the exposure is small and man made. Results are therefore not as relevant to this exposure as they are to coastal cliffs. The data used in the analysis are presented in Table 6.5, Figures 6.17-6.26 and Plates 6.3-6.12.

6.4.2.2 Results and Discussion

For each site a Factor of Safety is presented (Table 6.6-6.15). These are based on data collected specifically for this slope stability analysis and those discontinuities identified from the stereonets that form failure planes. Some general conclusions can be proposed but it may be noted that the results are not entirely as expected. The hypothesis that Factors of Safety values decrease from east to west, indicated by the increasing possibilities of instability shown by the field discontinuity pattern, does not seem to be valid according to these results. All Factors of Safety are much lower than expected and although individual results do vary, they suggest that all the coastal cliffs are unstable. Similar situations have been recognised by Jaeger & Cook (1979), who identify numerous slopes at angles greater than the angle of internal friction and the importance of parameters such as joint density, which is not considered here.

At all sites the cliffs plunge directly into the sea. An attempt was therefore made to determine the Factors of Safety, given saturation of the material at the base of the cliff and pore water pressures generated by the shock pressures of breaking waves. Under these conditions all but three of the Factors of Safety drop to zero. Clearly

TABLE 6.5

Data used in the stability analysis of Portland Limestone cliffs

| | Tillywhim | Seacombe | Winspit | Pondfield | Worbarrow Tout | Mupe | Fossil Forest | Lulworth Cove | Stair Hole | Durdle Door |
|---|---|---|---|---|---|---|---|---|---|---|
| Free Face Angle (°) | 90 | 90 | 85 | 80 | 66 | 58 | 66 | 68 | 71 | 74 |
| Dip of Bedding (°) | 0 | 0 | 0 | 36 | 38 | 28 | 33 | 41 | 42 | 85 |
| Corner angle of Basement Block (°) | 90 | 90 | 90 | 95 | 85 | 84 | 83 | 96 | 76 | 53 |
| Length of first block (m) | 1.58 | 0.61 | 0.72 | 1.95 | 1.38 | 1.12 | 1.32 | 1.28 | 1.23 | 1.27 |
| Length of second block (m) | 1.82 | 0.72 | 0.75 | 1.37 | 1.43 | 1.73 | 1.08 | 1.04 | 1.08 | 1.03 |
| Cliff height (m) | 28 | 37.5 | 43 | 99 | 58 | 36.5 | 43 | 27.5 | 55.5 | 29.5 |
| Total slope length (m) | 32 | 43.3 | 47 | 101 | 61 | 47.5 | 49 | 34 | 62 | 36 |

Note:

The above data were used in the Tetrahedral Block Analysis computer program, described in section 6.2.4. This is written in FORTRAN 80. Futher details cannot be presented here for commercial reasons, but can be obtained by writing to Geoffrey Walton, Consulting Mining and Engineering Geologists, Thames House, Market Street, Charlbury, Oxford, OX7 3PJ.

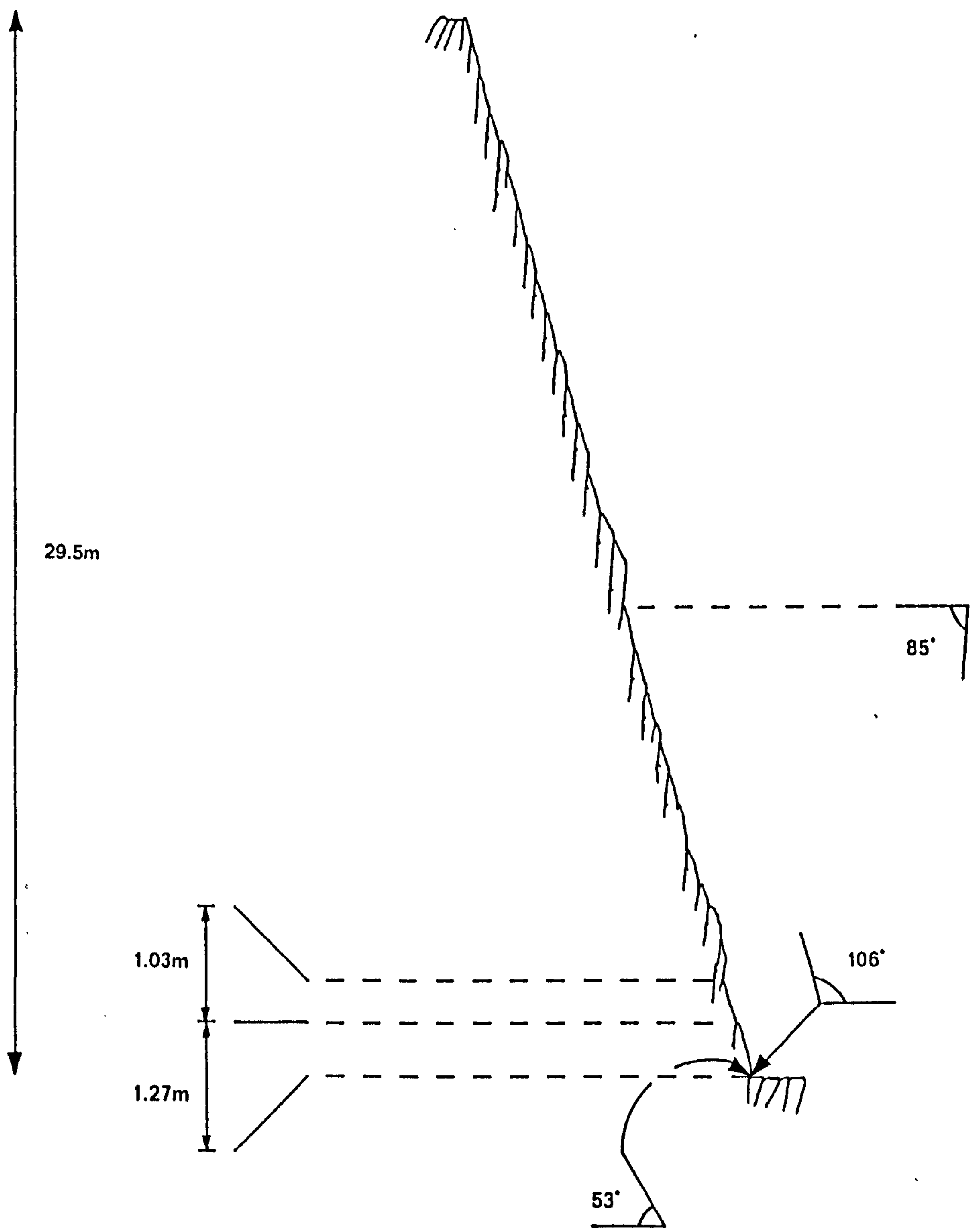



FIGURE 6.17

Details of the cliff section examined by stability analysis at Durdle Door

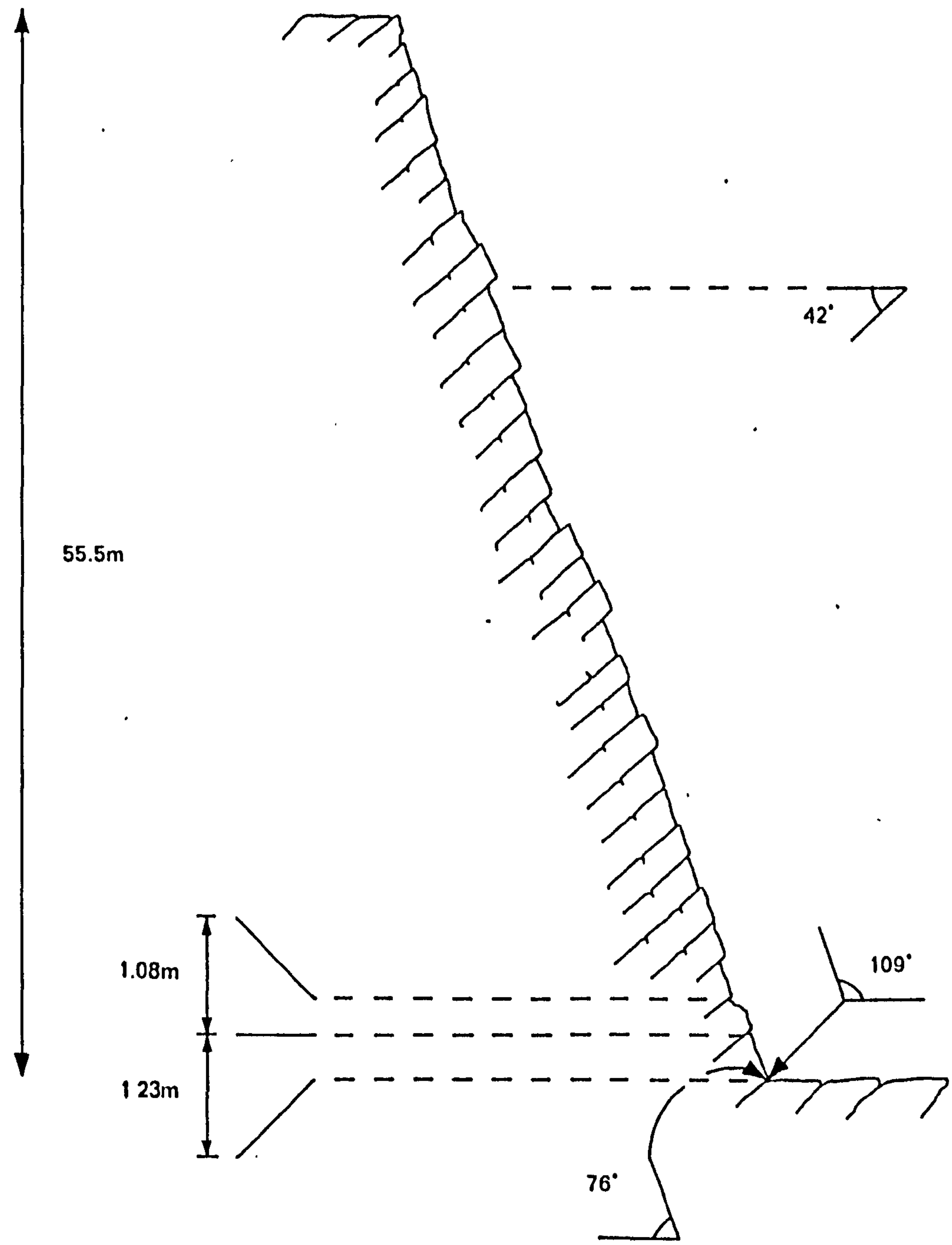



FIGURE 6.18

Details of the cliff section examined by stability analysis at Stair Hole

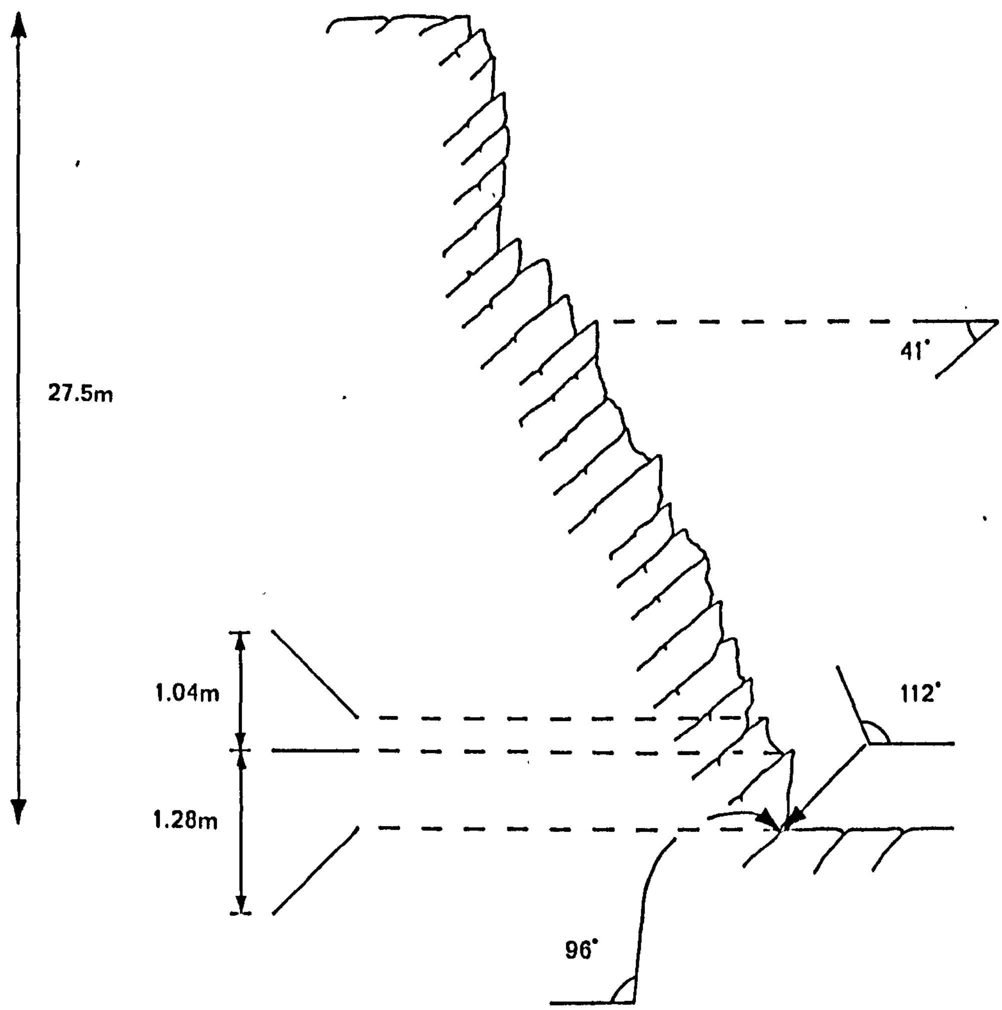



FIGURE 6.19

Details of the cliff section examined by stability analysis at Lulworth Cove

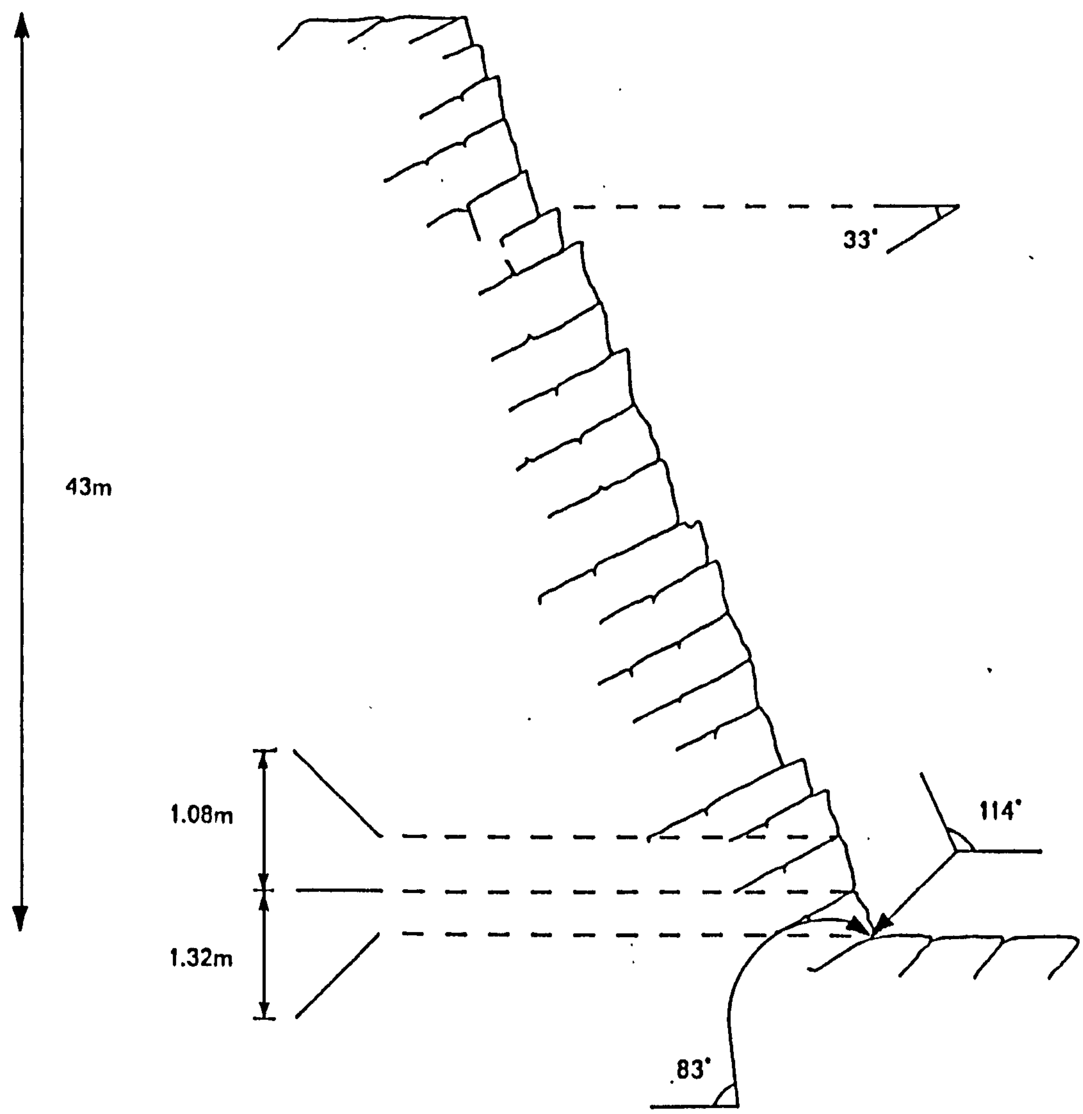



FIGURE 6.20

Details of the cliff section examined by stability analysis at Fossil Forest

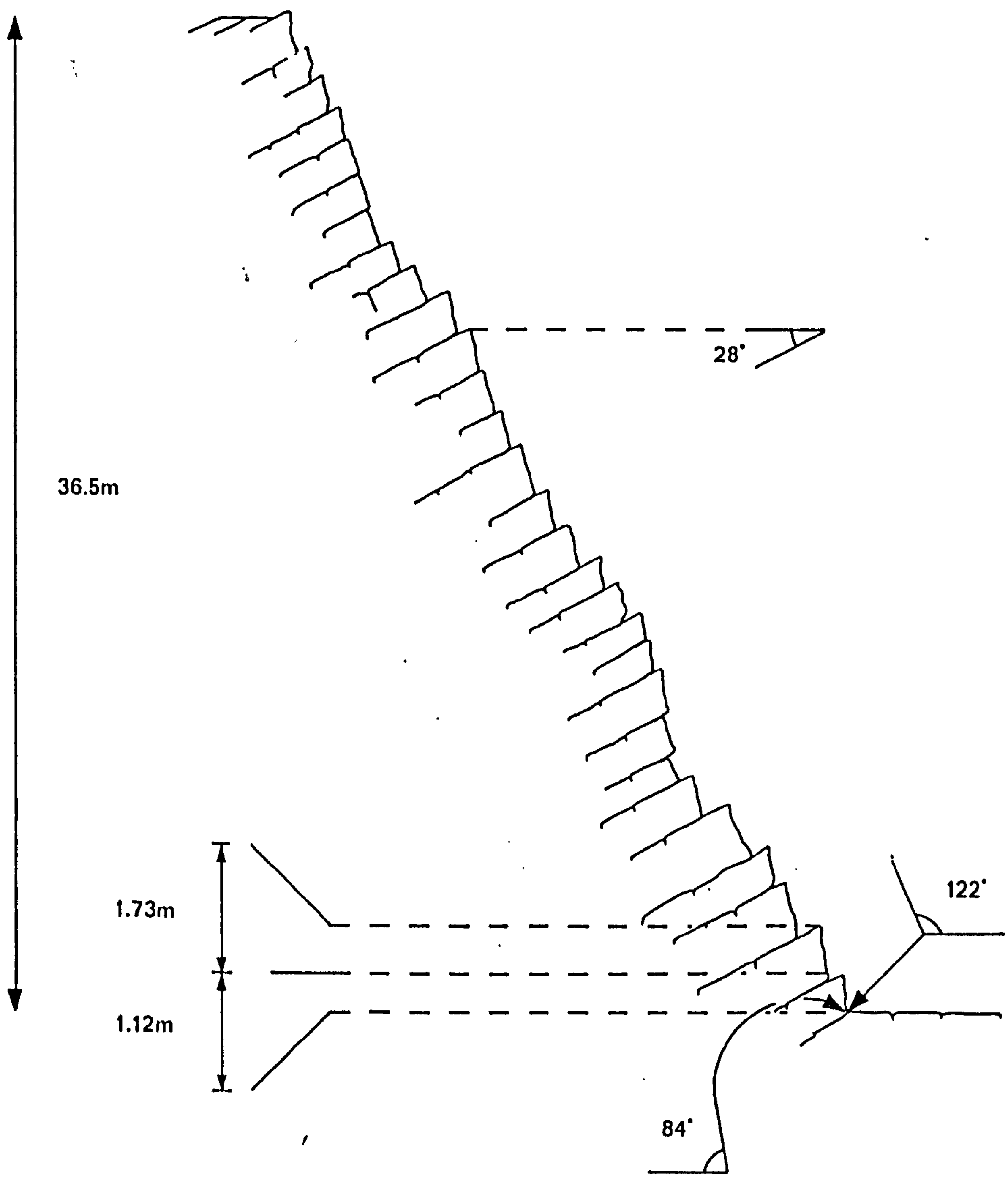



FIGURE 6.21

Details of the cliff section examined by stability analysis at Bacon Hole

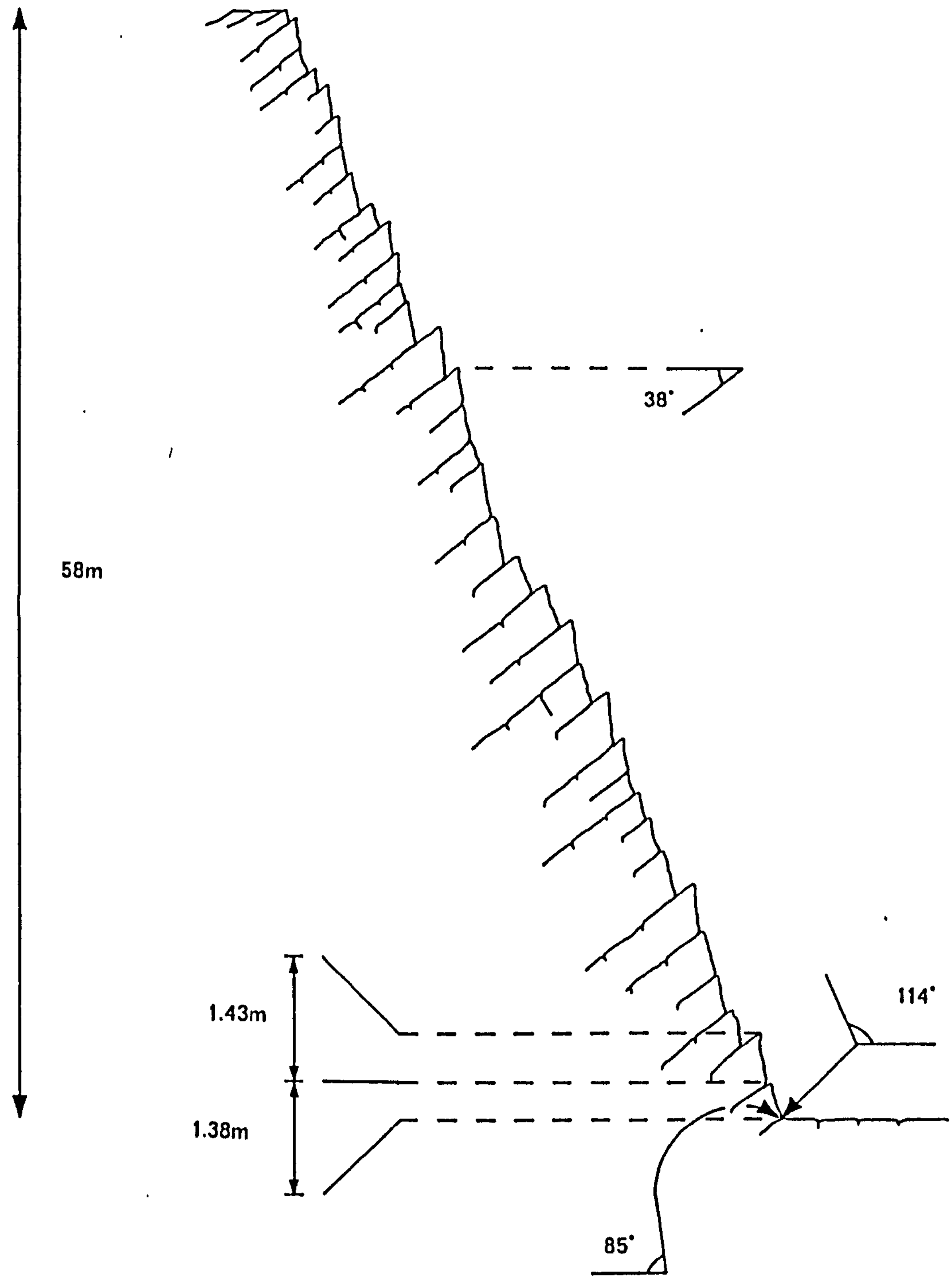



FIGURE 6.22

Details of the cliff section examined by stability analysis at Worbarrow Tout

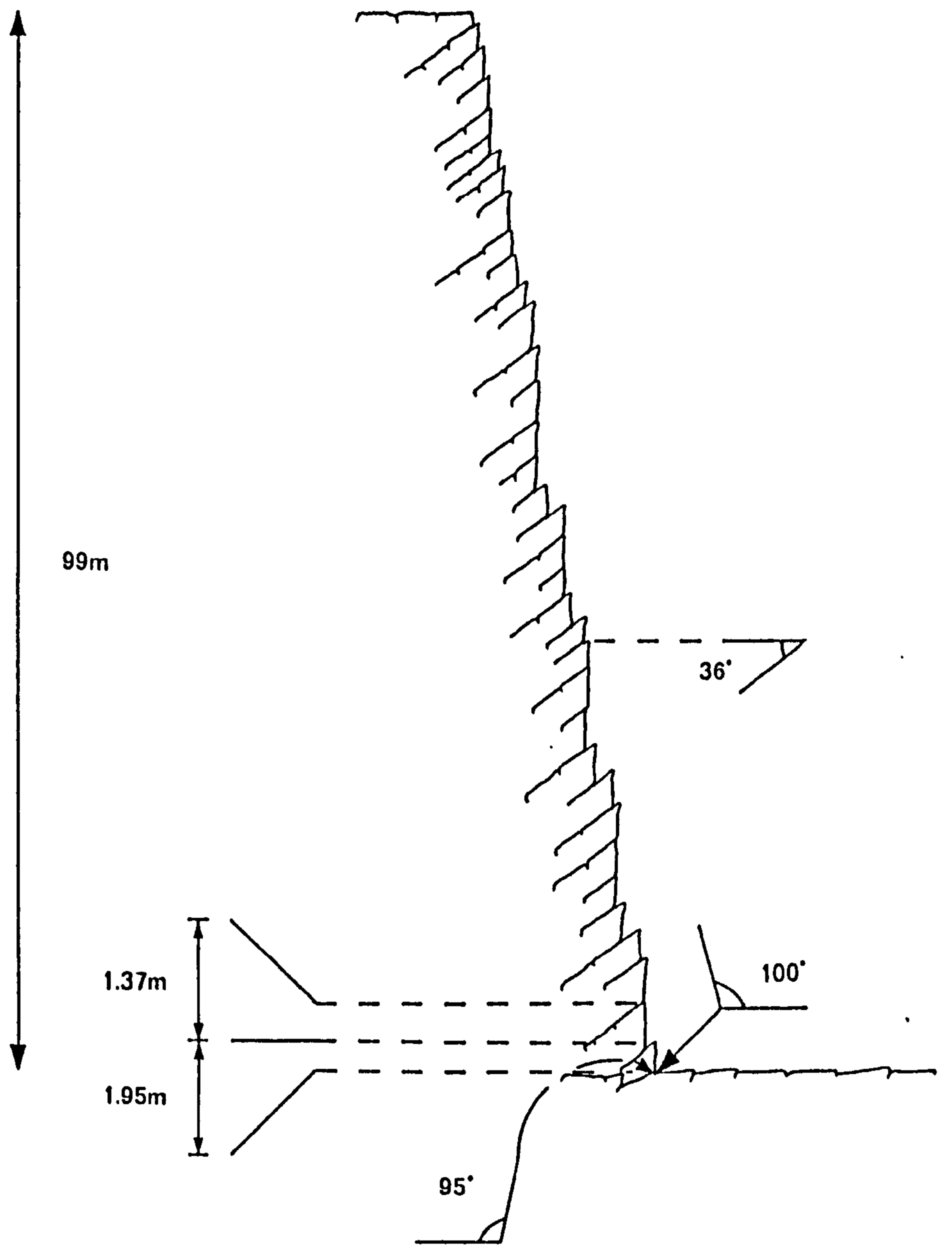



FIGURE 6.23

Details of the cliff section examined by stability analysis at Pondfield

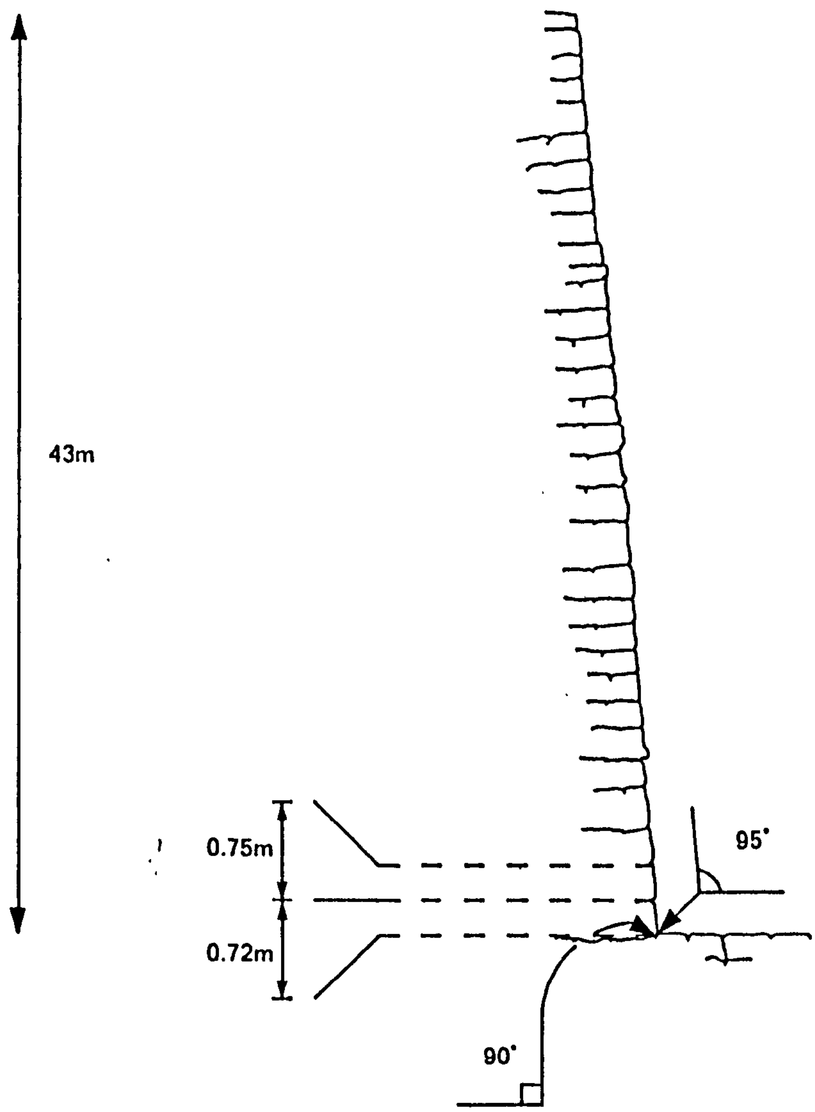



FIGURE 6.24

Details of the cliff section examined by stability analysis at Winspit

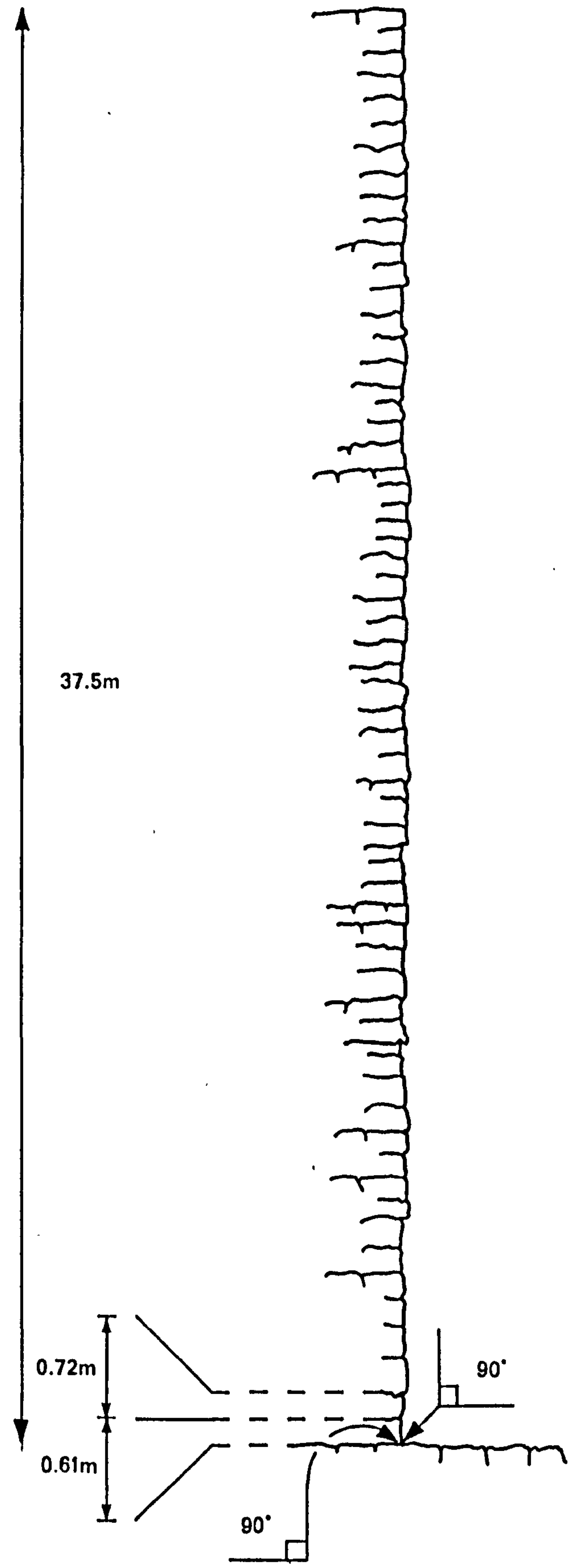



FIGURE 6.25

Detials of the cliff section examined by stability analysis at Seacombe

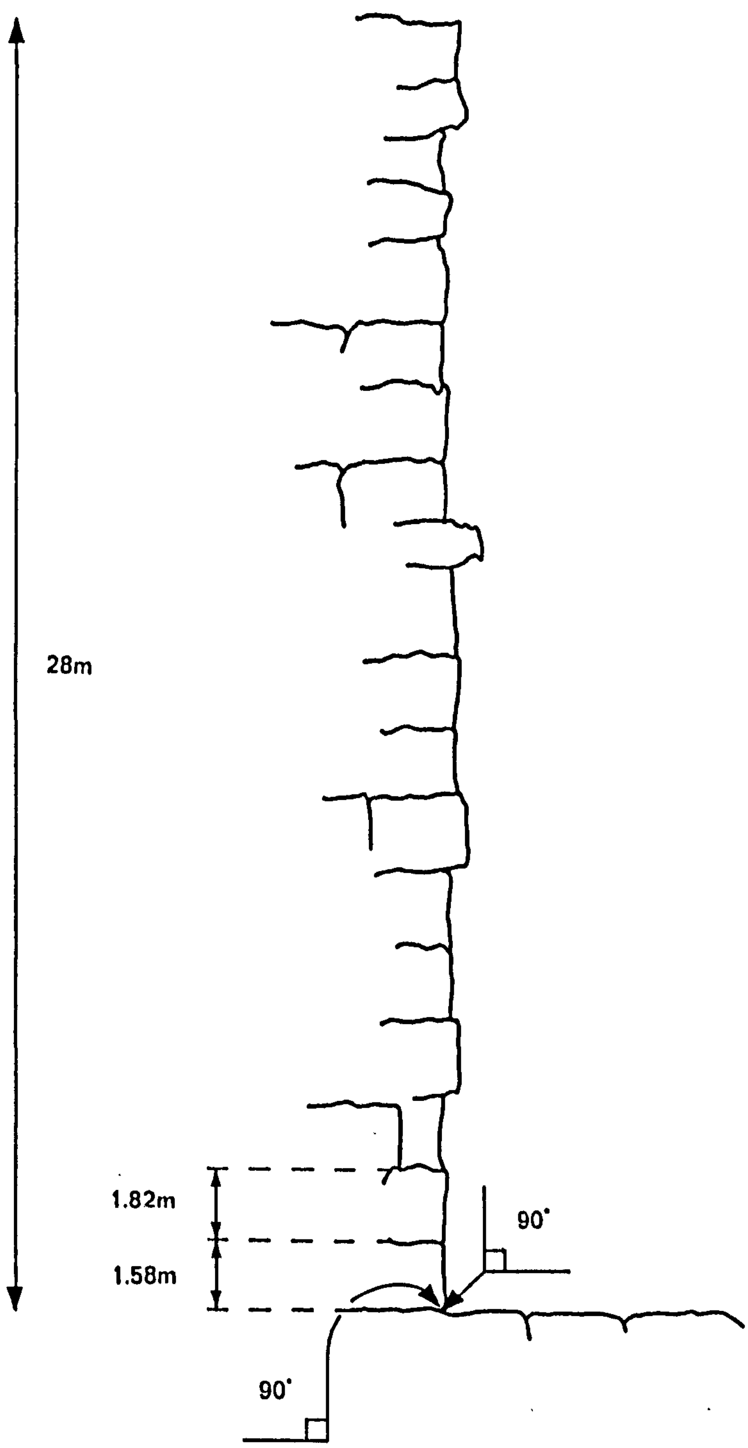



FIGURE 6.26

Details of the cliff section examined by stability analysis at Tillywhim

PLATE 6.3

Portland Limestone cliff section examined at Durdle Door

PLATE 6.4

Portland Limestone cliff section examined at Stair Hole

PLATE 6.5

Portland Limestone cliff section examined at Lulworth Cove

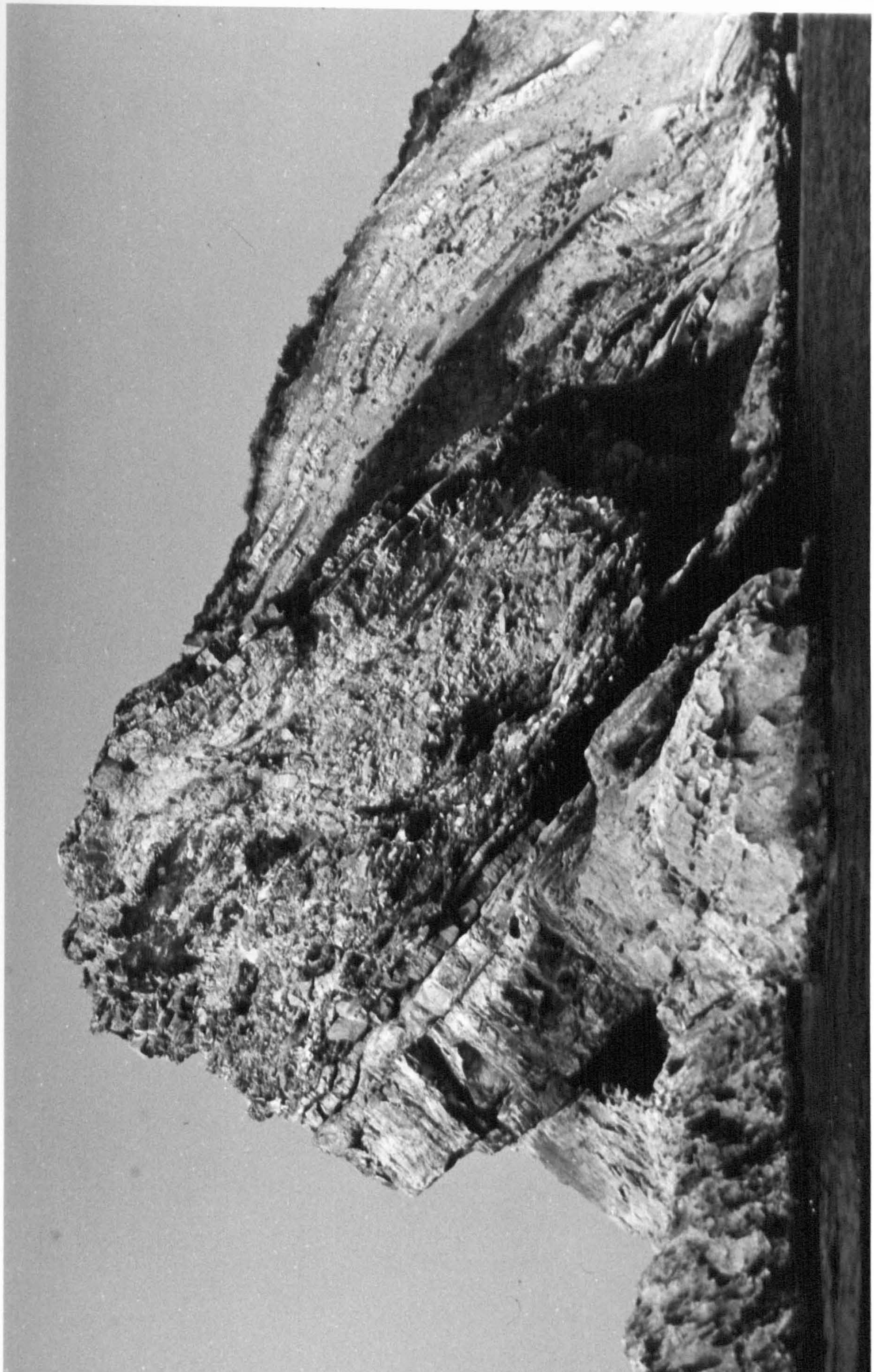


PLATE 6.6

Portland Limestone cliff section examined at Fossil Forest

PLATE 6.7

Portland Limestone cliff section examined at Bacon Hole

PLATE 6.8

Portland Limestone cliff section examined at Worbarrow Tout

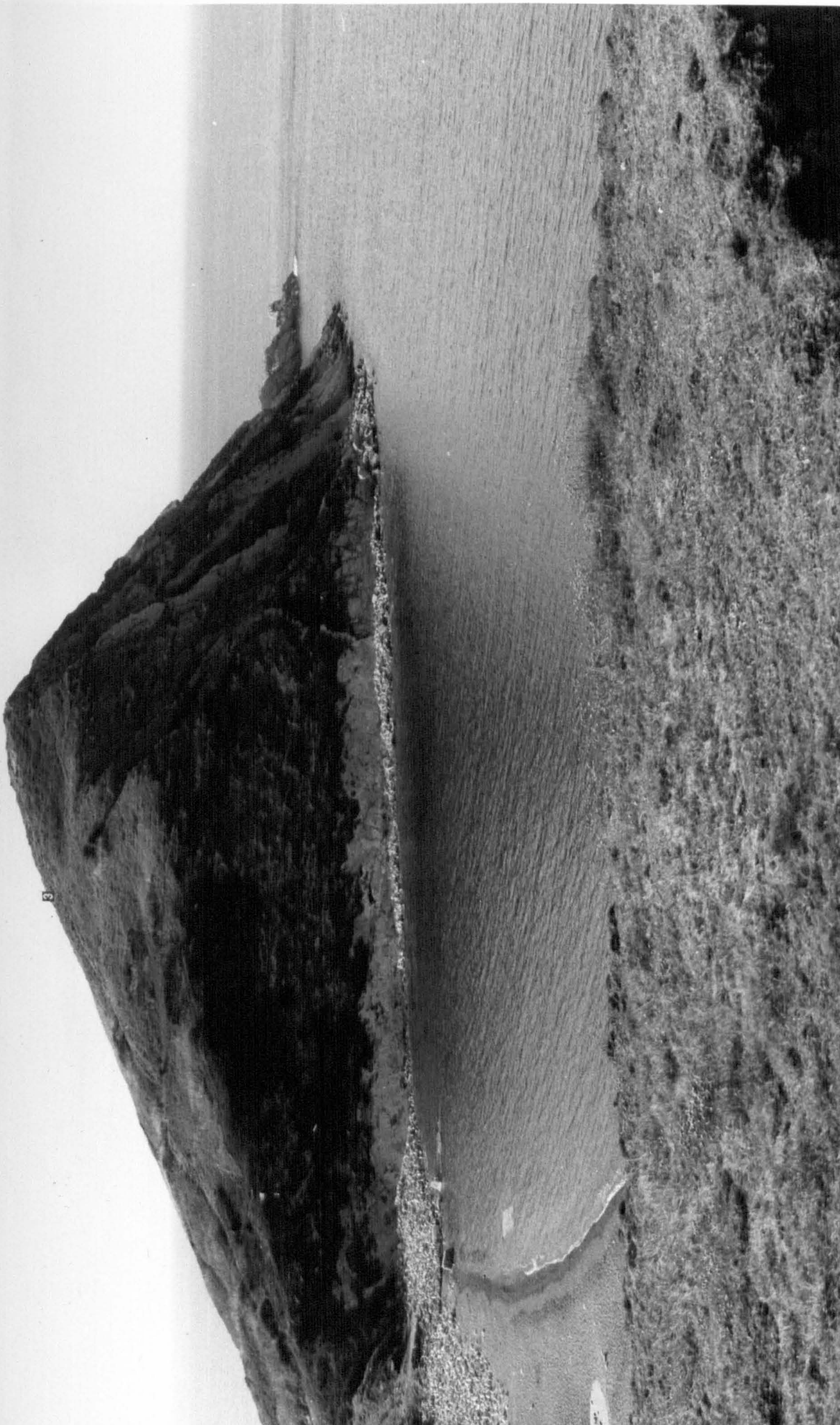


PLATE 6.9

Portland Limestone cliff section examined at Pondfield

PLATE 6.10

Portland Limestone cliff section examined at Seacombe

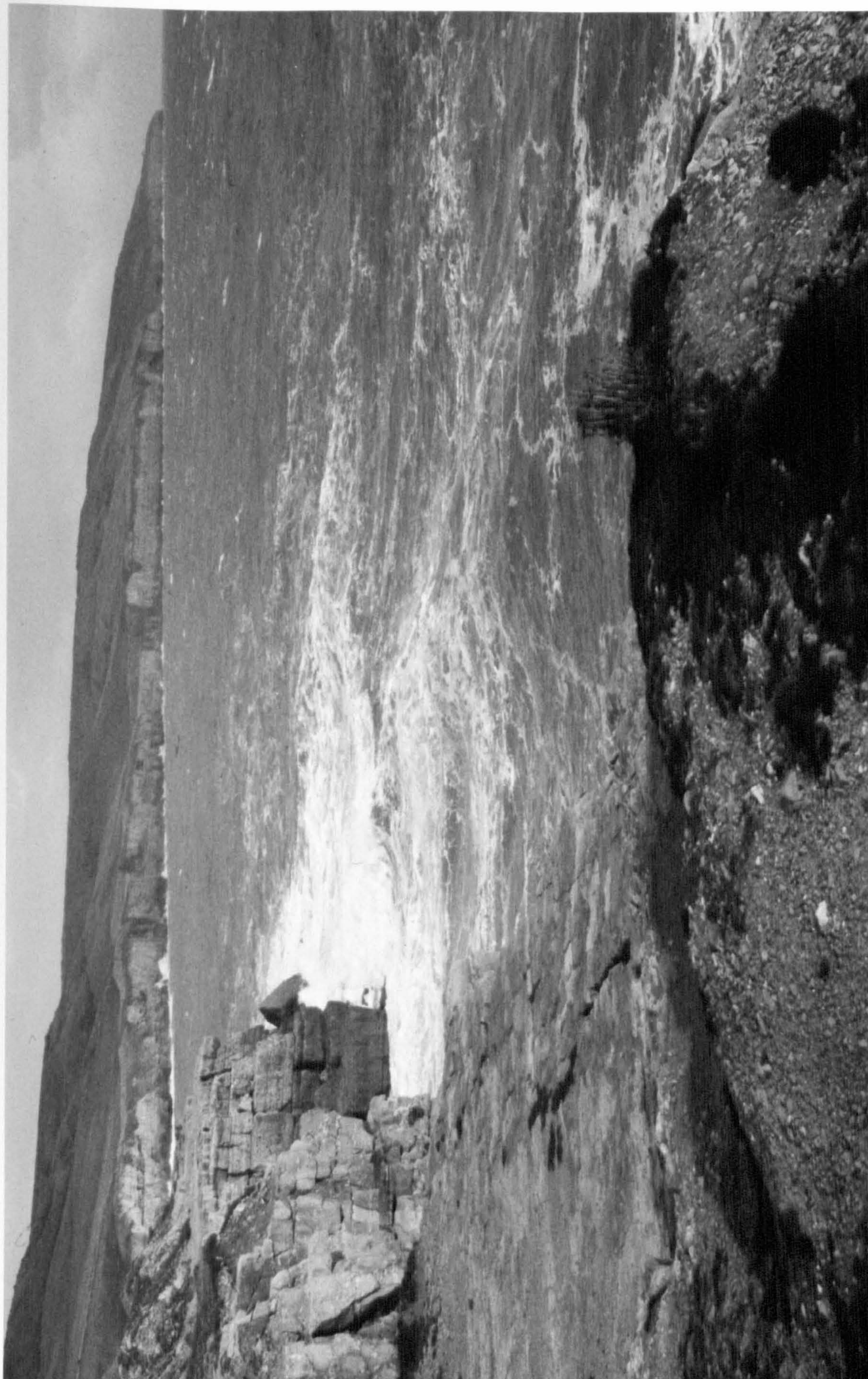
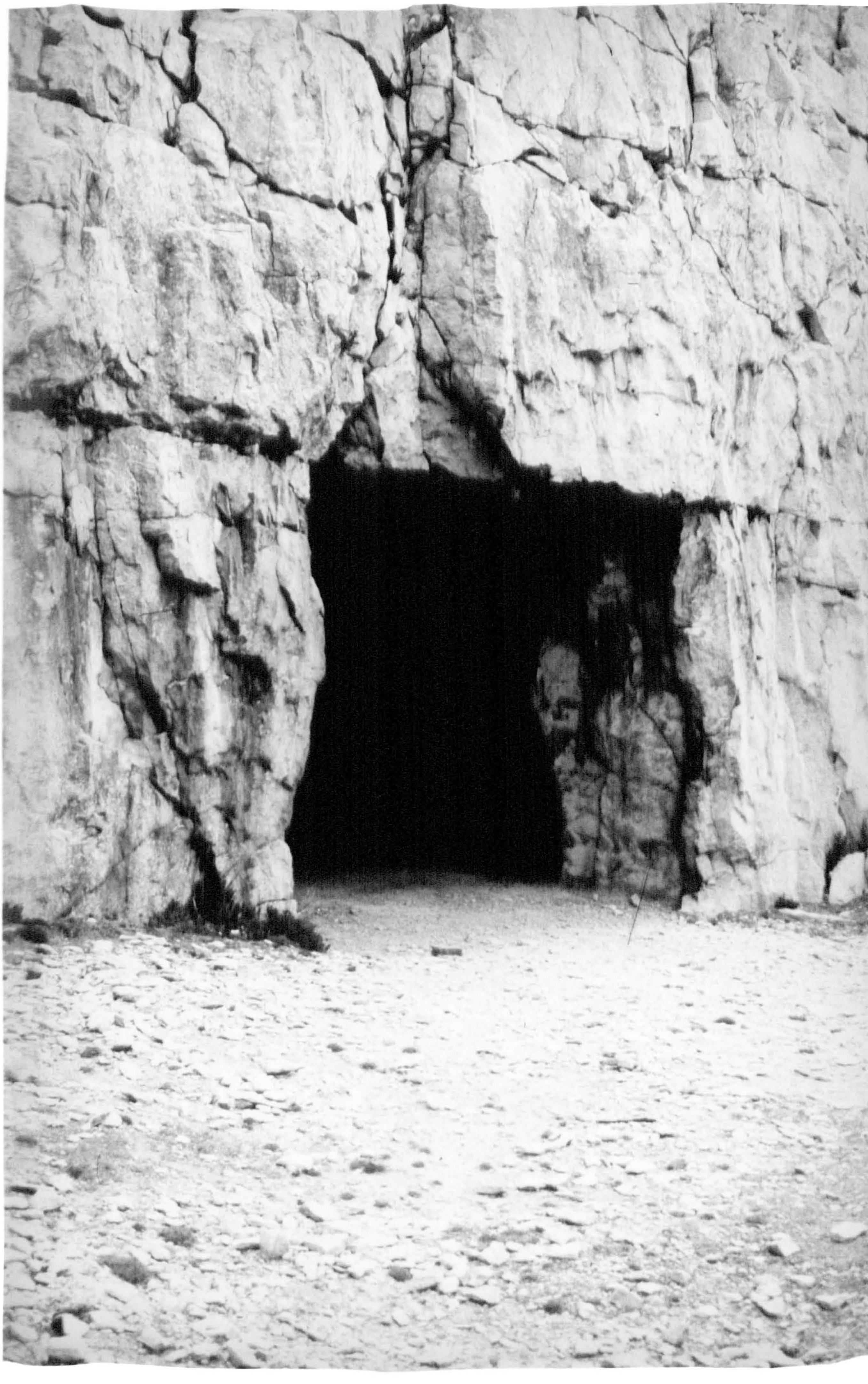


PLATE 6.11

Portland Limestone cliff section examined at Winspit

PLATE 6.12

Portland Limestone cliff section examined at Tillywhim



KEY FOR USE WITH TABLES 6.6 - 6.15

1. Potential Sliding Surface

This represents the mean orientation of those discontinuities responsible for failure and over which movement or sliding is most likely to occur. Details presented of each potential sliding surface include

i) Plane dip: Angle of dip of the discontinuity surface
ii) Orientation Dip Azimuth: Direction of the Azimuth of the discontinuity surface.

2. Where required, Plane Dip and Orientation Dip Azimuth are also included for the crest of the slope (Crestal Plane of Bench), the slope main free face (Facial Plane of Bench) and potential Tension Cracks which are likely to develop and influence the Factors of Safety.

3. Bench Height.

Represents total cliff height from toe to crest.

4. Distance to Tension Crack.

Represents details of position of potential Tension Cracks likely to be of importance relative to the cliff overall free face.

5. Density.

Represents measured material bulk density.

6. Line of Intersection of Potential Shear Surface.

Details of orientation of intersection of failure surfaces relative to details of the potential sliding surfaces listed previously.

7. Factors of Safety.

In each instance two factors of safety are listed.

i) With zero water pressure. In this instance it is assumed that the influence of marine processes is negligible on overall cliff stability, relative to those other processes responsible for the stability of the cliff.

ii) With a water pressure component thought to suitably represent basal situation of that section of cliff which is known to lie below mean sea level.

TABLE 6.6

Portland Limestone Stability Analysis

Durdle Door

Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 50.00 | 185.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 66.00 | 111.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 74.00 | 182.00 | | |
| Potential Tension Crack | 85.00 | 2.00 | | |

Bench Height 29.50 m

Distance to Tension Crack 0.00 m (measured along trace of plane 1)

Density 2.40 $Mg/m^3$

Line of Intersection of Potential Shear Surfaces:

Dip 49.04 Dip Direction 170.14

Water Pressure 0.00 kPa

Factor of Safety 0.67 (contact on both planes)

Water Pressure 48.22 kPa

Factor of Safety 0.21 (contact on both planes)

TABLE 6.7

Portland Limestone Stability Analysis

Stair Hole

Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 44.00 | 192.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 65.00 | 262.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 90.00 | | |
| Facial Plane of Bench | 68.00 | 198.00 | | |
| Potential Tension Crack | 44.00 | 18.00 | | |

| | |
|---|---|
| Bench Height | 27.50 m |
| Distance to Tension Crack | 0.00 m (measured along trace of plane 1) |
| Density | 2.40 $Mg/m^3$ |

Line of Intersection of Potential Shear Surfaces:

Dip 43.81 Dip Direction 198.57

| | |
|---|---|
| Water Pressure | 0.00 kPa |
| Factor of Safety | 0.76 (contact on both planes) |
| Water Pressure | 44.95 kPa |
| Factor of Safety | 0.27 (contact on both planes) |

TABLE 6.8

Portland Limestone Stability Analysis

Lulworth Cove

Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 61.00 | 167.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 83.00 | 100.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 71.00 | 182.00 | | |
| Potential Tension Crack | 32.00 | 2.00 | | |

| | |
|---|---|
| Bench Height | 29.50 m |
| Distance to Tension Crack | 0.00 m (measured along trace of plane 1) |
| Density | 2.40 $Mg/m^3$ |

Line of Intersection of Potential Shear Surfaces:

Dip 60.59 Dip Direction 177.42

| | |
|---|---|
| Water Pressure | 0.00 kPa |
| Factor of Safety | 0.48 (contact on both planes) |
| Water Pressure | 48.22 kPa |
| Factor of Safety | 0.00 (contact lost on both planes) |

TABLE 6.9

Portland Limestone Stability Analysis

Fossil Forest

## Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | Shear Strength PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 64.00 | 175.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 80.00 | 67.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 68.00 | 180.00 | | |
| Potential Tension Crack | 36.00 | 0.00 | | |

Bench Height 43.00 m

Distance to Tension Crack 0.00 m (measured along trace of plane 1)

Density 2.40 $Mg/m^3$

Line of Intersection of Potential Shear Surfaces:

Dip 59.17 Dip Direction 139.81

Water Pressure 0.00 kPa

Factor of Safety 0.34 (contact on Plane 1 only)

Water Pressure 70.28 kPa

Factor of Safety 0.00 (contact lost on both planes)

TABLE 6.10

Portland Limestone Stability Analysis

Bacon Hole

Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 75.00 | 110.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 70.00 | 236.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 58.00 | 174.00 | | |
| Potential Tension Crack | 28.00 | 354.00 | | |

Bench Height 43.00 m

Distance to Tension Crack 0.00 m (measured along trace of plane 1)

Density 2.40 $Mg/m^3$

Line of Intersection of Potential Shear Surfaces:

Dip 55.08 Dip Direction 177.43

Water Pressure 0.00 kPa

Factor of Safety 0.92 (contact on both planes)

Water Pressure 70.28 kPa

Factor of Safety 0.00 (contact lost on both planes)

TABLE 6.11

Portland Limestone Stability Analysis

Worbarrow Tout

Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 57.00 | 157.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 76.00 | 245.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 66.00 | 161.00 | | |
| Potential Tension Crack | 37.00 | 341.00 | | |

Bench Height 58.00 m

Distance to Tension Crack 0.00 m (measured along trace of plane 1)

Density 2.40 $Mg/m^3$

Line of Intersection of Potential Shear Surfaces:

Dip 55.48 Dip Direction 176.25

Water Pressure 0.00 kPa

Factor of Safety 0.58 (contact on both planes)

Water Pressure 94.80 kPa

Factor of Safety 0.00 (contact on Plane 2 only)

TABLE 6.12

Portland Limestone Stability Analysis

Pondfield

## Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 48.00 | 148.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 77.00 | 235.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 80.00 | 160.00 | | |
| Potential Tension Crack | 39.00 | 340.00 | | |

Bench Height 99.00 m

Distance to Tension Crack 0.00 m (measured along trace of plane 1)

Density 2.40 $Mg/m^3$

Line of Intersection of Potential Shear Surfaces:

Dip 47.42 Dip Direction 159.55

Water Pressure 0.00 kPa

Factor of Safety 0.72 (contact on both planes)

Water Pressure 161.82 kPa

Factor of Safety 0.31 (contact on both planes)

TABLE 6.13

Portland Limestone Stability Analysis

Winspit

Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 79.00 | 179.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 81.00 | 85.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 85.00 | 135.00 | | |
| Potential Tension Crack | 85.00 | 262.00 | | |

Bench Height 20.00 m

Distance to Tension Crack 0.00 m (measured along trace of plane 1)

Density 2.40 $Mg/m^3$

Line of Intersection of Potential Shear Surfaces:

Dip 75.44 Dip Direction 137.44

Water Pressure 0.00 kPa

Factor of Safety 0.26 (contact on both planes)

Water Pressure 34.69 kPa

Factor of Safety 0.00 (contact lost on both planes)

TABLE 6.14

Portland Limestone Stability Analysis

Seacombe

Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 79.00 | 193.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 86.00 | 95.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 90.00 | 166.00 | | |
| Potential Tension Crack | 87.00 | 279.00 | | |

Bench Height 20.00 m

Distance to Tension Crack 0.00 m (measured along trace of plane 1)

Density 2.40 $Mg/m^3$

Line of Intersection of Potential Shear Surfaces:

Dip 77.62 Dip Direction 166.26

Water Pressure 0.00 kPa

Factor of Safety 0.21 (contact on both planes)

Water Pressure 32.69 kPa

Factor of Safety 0.00 (contact lost on both planes)

TABLE 6.15

Portland Limestone Stability Analysis

Tillywhim

## Wedge Parameters

| | Plane Dip | Orientation Dip Azimuth | Shear Strength C(kpa) | PHI |
|---|---|---|---|---|
| 1st Potential Sliding Surface | 78.00 | 181.00 | 0.00 | 35.00 |
| 2nd Potential Sliding Surface | 85.00 | 76.00 | 0.00 | 35.00 |
| Crestal Plane of Bench | 0.00 | 0.00 | | |
| Facial Plane of Bench | 90.00 | 140.00 | | |
| Potential Tension Crack | 64.00 | 259.00 | | |

Bench Height 28.00 m

Distance to Tension Crack 0.00 m (measured along trace of plane 1)

Density 2.40 $Mg/m^3$

Line of Intersection of Potential Shear Surfaces:

Dip 75.49 Dip Direction 146.24

Water Pressure 0.00 kPa

Factor of Safety 0.28 (contact on both planes)

Water Pressure 45.77 kPa

Factor of Safety 0.00 (contact lost on both planes)

a reduction in the relevant values can be expected but it is unlikely that in reality the change would be as great as this.

Nevertheless, attempts can be made to draw conclusions from the results and identify trends and spatial patterns from the data. These must be done bearing the above points in mind, however, and also must assume that although the results appear to be particularly low, this is uniformly maintained throughout the region, permitting relative comparisons.

Factors of Safety are much lower at Tillywhim (Figure 6.26, Plate 6.12, Table 6.15), Seacombe (Figure 6.25, Plate 6.11, Table 6.14) and Winspit (Figure 6.24, Plate 6.10, Table 6.13) than elsewhere along the coast. Field evidence runs contrary to this, however, with both undisturbed cliff morphology, evidence of recent rockfalls and the presence of quarrying at these three locations all suggesting that the overall results should be higher. The isolation of these three sites due to the low Factors of Safety does, however, conform to the 'A' grouping based on the discontinuities and identified from the stereographic projections. It is therefore suggested that, due to the regional structure, the discontinuity characteristics are of greater significance to cliff stability than might be expected. The anomaly between the results from the stereonet analyses and the stability analyses, with the former suggesting stable cliffs and the latter unstable cliffs requires some explanation. At Tillywhim, Seacombe and Winspit, as previously discussed, the discontinuity pattern forms cuboidal blocks. Assuming completely regular horizontal and vertical jointing,the blocks would lie in columns and be particularly susceptible to failure, as suggested by the Factors of Safety. In reality, however, although specific joint sets can be identified, there is some variability within each group. This seems to create particularly stable cliffs under field conditions. Available slope stability models do not currently have the capacity to take such characteristics into account however.

The remaining Factors of Safety conform to group 'B' locations defined from the stereonets. There appears to be no significant or explicable trend to the results, with variations in the Factors of Safety between 0.92 at Bacon Hole (Figure 6.21, Plate 6.7, Table 6.10) and 0.34 at the Fossil Forest (Figure 6.20, Plate 6.6, Table 6.9). However, a number of points can be noted. The Factors of Safety results do not suggest a subdivision of these sites into groups B(i) and B(ii), as suggested by the analyses of the discontinuity data. The Factor of Safety of 0.92 at Bacon Hole is significantly higher than at the other sites but there is no apparent reason why this should be so. If this particular result is excluded, the remaining Factor of Safety values do show a tendency to be lower towards the central area of the cliff-line. Again, however, acceptable explanation of this trend cannot be identified.

An attempt to simulate marine effects on the stability of all slopes does not provide particularly useful results. In all instances the Factors of Safety have dropped, but due to their already low values, they are zeroed in all but three cases. Values this low are highly unlikely to occur in the landscape. However where numeric results are present under these modified hydrological conditions they are at least from the sites where the Factor of Safety values for unsaturated conditions are higher.

In conclusion, these results have to be viewed with caution, assuming underestimation in the Factors of Safety. It is likely that this is because the program requirements do not include all details critical to the stability of the slope, such as joint friction, surface roughness, joint density and rock strength (Terzaghi, 1962; Hoek, 1983). However, it is reasonable to suggest that the program has been successful if it has identified the critical parameters to be measured in future research, provided suggestions for further refining currently available models and permitted between-site comparisons. It is therefore necessary to state that if the principal use of Slope Stability Analysis is to examine differences in the potential hazard along the coast due to variations in the cliffs, such conclusions cannot be confidently made for the Portland Limestone in the Isle of Purbeck.

Selby (1981) suggested that "detailed field observations may be more valuable in interpreting or predicting the causes or possibilities of landslides in rocks". This seems to hold true in this instance. Selby (1982) further suggests that the investigation of geological structures, their associated parameters and hillslope forms may be "a better guide (to slope stability) than much study in the laboratory or much calculation". Again this seems to be the case here and with particular reference to the Portland Limestone, as Morgenstern & Sangrey (1978) suggest, "analysis of a rock slope in terms of a Factor of Safety is a subordinate activity to achieving a clear understanding of the controlling geology".

## 6.5 CONCLUSIONS

The stability analysis conducted as part of this study has provided much useful additional information regarding the mass movements and development of the coastal cliffs in the Isle of Purbeck. Despite initial problems with some of the data, a large number of different results have been obtained, giving a clearer picture of the effects of variations in important material properties, field conditions and failure mechanisms. Of particular success has been the assimilation of the results of the field mapping, geotechnical investigation and field monitoring programs.

The analyses have also identified a number of areas where further investigations are required, both relative to the techniques available for analysing the stability of slopes and their application to this specific area. Results show that on occasions anomalous conclusions can be reached, if a study such as this is limited in its scope to one aspect of the overall investigation. The inconclusive results also raise the worrying question of whether the critical parameters controlling the cliffs have been identified. If the analyses had produced Factors of Safety which were above or close to unity, the temptation would have been to accept them and to base further research conclusions upon them. This subject is a vital area for future research on the validity of available techniques.

Finally it should be noted, as stated at the beginning of this chapter, that stability analysis does not assume the focal point of the overall study. It is rather an outstandingly useful investigative tool, assimilating and summarising the findings of a large part of this research and providing a particularly useful umbrella beneath which important statements can be made about the coastal cliffs.

# CHAPTER VII CONCLUSIONS

This thesis has presented the results of an integrated study of mass movements along the Isle of Purbeck coastline. Particular attention has been paid to the cliffs between Worbarrow Tout and Durdle Door.

The aims of the study were to complete a comprehensive investigation of recent spatial and temporal changes in the cliffs and hence assess coastline development; identify links between geology and geomorphology; classify the mass movements and their causal mechanisms; conduct a thorough geotechnical investigation of the important rock units; examine the detailed mechanisms of mudslides, since they form the most common type of mass movement in this area; improve investigatory procedure for this type of study, by developing new equipment and applying techniques which have not previously been utilised in similar work and assess the current stability of these coastal cliffs and their potential for failure.

This study includes the following work that makes an original contribution to knowledge.

A detailed geomorphological survey of the coastal landforms between Durdle Door and Worbarrow Tout has been conducted (chapter III). The mass movement types which are present have been identified, classified (section 3.3.2.1) and their spatial relationships have been examined (section 3.3.2.2). In the Wealden Beds active rotational slides are found only at Worbarrow Bay, while mudslides occur at all coastal sites in this lithology (section 3.4.2).

Mudslide size decreases from east to west, as the width of the Wealden outcrop is reduced. The position of individual mass movements is significantly controlled by variations in the stratigraphy (section 2.3). The reduced outcrop width and the stratigraphic complexity prevents the development of rotational slides at Mupe Bay, Stair Hole and Durdle Door. Mudslides frequently develop along the axis of argillaceous bands between harder, cemented, sandy units, which are resistant to weathering and erosion. Between these discrete features, areas of translational sliding and shallow surface movements are found. Soil creep is also evident on slopes covered in vegetation.

In the Portland Limestone, block detachment, wedge failures and topples occur at different points along the coastline (section 3.3.2.2). A transition occurs from block detachment in the east where bedding is flat, to wedge failures where bedding dips to the north at low angles (sections 4.4.5 and 6.4.1). A combination of toppling and wedge failures is predominant in the west of the study area, where bedding dips steeply. Similar spatial trends are much more difficult to identify in the Chalk. This is due to the structural complexity of this rock unit (section 2.4) and variations in the measured geotechnical properties in different parts of the outcrop (section 4.4.1).

Changes in the cliff line over the past century have been identified and spatial and temporal trends in development established (chapter III). Some cliffs are clearly retreating more rapidly than others, both between and within different rock units (section 3.3).

Coastline retreat has been most rapid where the Wealden Beds crop out. The cliffs formed in this material at Worbarrow Bay have experienced the greatest change. Rockfalls in the Chalk and Portland Limestone do not necessarily result in the loss of sections of stable ground. Mass movements in these materials have consequently been more frequent than is indicated from this type of investigation. Most detachment has occurred from the free face, and changes in the location of the crest of the slope do not accurately reflect either the magnitude or the frequency of failure events.

Temporal changes in the position of the cliff edge, established from 100 year cartographic data (section 3.3.1), show that many cliff sections developed in Chalk and Portland Limestone have displayed no change since the earliest record in 1888. Conversely, many of the cliff sections in the Wealden Beds have undergone retreat. This is evident on more recent map editions, suggesting not only spatial but also temporal trends.

A number of individually identifiable areas of rapid cliff retreat can be seen on the eastern and western flanks of Lulworth Cove. This reflects the significance of the Purbeck/Wealden junction in influencing local mass movements.

The importance of the Purbeck monocline in relation to the type, size and distribution of mass movements has been established (chapter IV). The location, type and activity of the mass movements are influenced to a significant degree by their geological setting. As the width of the Wealden outcrop decreases, so does the surface area of individual mudslides. These mass movements are large at Worbarrow Bay, becoming gradually smaller to the west. The structural control is clearly seen in the Portland Limestone. In eastern Purbeck, the flat-lying limb of the fold displays horizontal bedding. Block detachment is the dominant failure mechanism in this area. Between Emmett's Hill and Stair Hole bedding dips to the north, at angles of around 30°N (section4.4.5). Wedge failures accompany this disposition. In the far west, at Durdle Door, the bedding dips at almost 90° and is accompanied by both topples and wedge failures.

The Chalk in Purbeck is structurally complex. In fact, it is difficult to identify any one of a number of structural features as being of outstanding importance to cliff stability and failure type. Thrusts, fault planes, varied discontinuity patterns, stylolites and variations in microstructure (section 4.4.4) are all important.

A thorough geotechnical survey of the Portland Limestone, Wealden Beds and Chalk (chapter IV) has been conducted. This is the first time these materials have been subjected to detailed investigation. Variations in material properties for the Portland Limestone do not follow a marked regular trend (sections 4.4.2 - 4.4.5). This is highlighted by the results of field tests using the Schmidt Hammer. Some important variations can be recognised, however, with greatest material mechanical competence being seen at Durdle Door and favourable material characteristics at Stair Hole, Lulworth Cove and Worbarrow Tout. The remaining sites comprise relatively weak sediments, with the cliffs at Winspit displaying particularly unfavourable characteristics.

Results for the Chalk display important regular spatial changes which undoubtedly exert a significant control on the stability of the coastal cliffs (sections 4.4.1 and 4.4.3 - 4.4.5). These are best explained with reference to the Purbeck monocline. Stronger, mechanically more competent materials, are found in the north-east with the weakest materials outcropping in the south-west. The changes in material properties are reflected in resultant topography with low cliffs at Studland and particularly steep, high cliffs at Worbarrow Bay, for example. Variations in the discontinuity pattern of the Portland Limestone depend on the point within the overall structure at which the cliff section occurs (section 4.4.5.2). From east to west along the coast, bedding changes from horizontal to almost vertical and joints swing from a dominant east-west/ north-south trend to a north-east, south-west/south-east, north-west direction. The failure mechanisms in the Portland Limestone relate to these changes in fracture pattern.

The geotechnical characteristics of the Wealden Beds (section 4.5) suggest that the outcrop at Durdle Door is clearly different from other coastal cliffs formed in this material. Strength tests (section 4.5.1) show that the Wealden outcrop at Durdle Door displays greater material competence, while bulk density values (section 4.5.2) are also high at this site and the particle size distribution indicates a large sand content (section 4.5.3) if the results are compared with those for the other outcrops. Material properties at Mupe and Worbarrow Bays suggest that these two outcrops bear close similarities in many of their properties.

A rock classification has been developed to provide a summary of the material properties important to cliff stability (section 4.6). Many previous studies of a similar nature have merely summarised geotechnical investigations, with a broad statement based on the individual results and no attempt to assimilate the data. This study shows that by adopting a combined approach, a more accurate assessment of overall site characteristics can be made and potential anomolies arising from placing too much emphasis on one specific set of results do not occur. It is clear that the parameters in a classification must be dictated, to a considerable degree, by the type of material being examined and the use to which the results are to be put. This is highlighted by the need for two different classification systems, one for the Portland Limestone (section 6.4.1) and the other for the Wealden Beds (section 6.4.2). The Portland Limestone displays more favourable results, in terms of the likely effects on cliff stability, towards the western end of the study area. This is despite an increase in joint density, a parameter usually regarded as being unfavourable to slope stability. This indicates that other variables, in combination, counterbalance the increasingly unfavourable disposition of discontinuities from east to west.

Results for the Wealden Beds indicate that conditions most favour stability at Durdle Door. Intermediate results at Mupe Bay and Worbarrow are similar. Site conditions at Stair Hole are poor. These values broadly conform to expected trends but also highlight the need for field monitoring, to provide a comprehensive statement about the mass movements.

New details have been presented of movement mechanisms in mudslides (chapter V). Detailed monitoring at one site, in conjunction with broader investigations at four locations, have lead to a clearer understanding of mudslide movement and the parameters controlling instability. Seasonal cumulative mudslide movements (section5.6) have been broken down into four component parts. These include 'multiple' (stick slip), 'graded' and 'surge' events. Small, random displacements also occur as the mudslide material plastically deforms under the influence of gravity (section 5.7.4). These movements combine to produce the standard annual pattern of slip recorded in the general monthly sampling framework adopted for this study (section 5.6).

The frequency of each type of event varies. 'Multiple' (stick slip) events are small movements of no more than a few centimetres, occurring over a lapsed time of about 36 hours (section 5.7.1). They occur both individually and also as precursors to more rapid change. 'Graded' movements involve displacements of up to 0.5 m. typically occurring over 18 hours (section 5.7.2). 'Surges', on the other hand, are characteristically in excess of 2.0 - 3.0 m (section 5.7.3). They take place over a short lapsed time. In this study one 'surge' was recorded, with maximum displacements exceeding 3.0 m in 5 minutes.

Pore water pressures vary for each type of event. A small, gradual rise in the phreatic surface in the vicinity of the movement, causes 'multiple' (stick slip) displacements but there is no evidence of accompanying change to the phreatic surface at the toe of the slope. 'Graded' movements result from a distinct rise in the phreatic surface on the slope in proximity to the point where slip occurs. A slight rise in pore water pressure is also recorded at the toe of the slope. 'Surges' affect the phreatic surface along the whole mudslide. Pore water pressure rises rapidly at the point on the mudslide where movement is greatest. It also increases lower down the slope, due to the loading effect of material towards the top of the track.

The application of ultrasonic pulse velocity testing apparatus has been introduced to geomorphological research and the potential advantages have been clearly identified. The Grindosonic apparatus has been successfully utilised (chapter IV). This is a non-destructive technique, which can be used to determine Dynamic Young's Modulus as an indirect measure of rock strength, as well as a number of other parameters including Seismic Velocity and Poisson's Ratio (section 4.2.2.2). Samples can be subjected to multiple testing under different controlled conditions. In this study specimens were tested in both dry and saturated states, in an attempt to identify marine effects on material geotechnical properties. The results of tests conducted on Portland Limestone and Chalk samples using this equipment correlate well with data obtained using a standard Hoek Cell (sections 4.4.1 and 4.4.2). Relationships between Dynamic Young's Moduli, the porosity and density of the materials (sections 4.4.3 and 4.4.4) suggest that these parameters ought to form a significant part of any geotechnical investigation into these types of material. Much broader application of the Grindosonic technique can also be envisaged, in the study of environmental weathering, for example, where repeated testing of samples is required during laboratory simulation.

New electronic data logging apparatus has been designed, installed and tested (chapter V). Movements have been monitored using rotating potentiometers by attaching them to pins installed across the slope using nylon wire. Pulley wheels of precise dimensions were attached to the potentiometer shafts (section 5.4.5), to take-up spare wire and provide a direct linear relationship between slope movement and changes in the position of the variable resistor. Changes were recorded on magnetic tapes, through an interface designed specifically for this task. The equipment provided a detailed data set unsurpassed by previous studies (section 5.4.6).

The application of new techniques has also been assessed. The Neutron Probe (section 5.4.3) permitted the determination of complete soil moisture profiles at a number of locations on the instrumented mudslide, rather than giving point readings which would have been restricted in both time and three-dimensional space (section 5.6.3). A Hammer Seismograph was successfully used to identify the slip surface of mudslides (section 5.3.2). The results were cross-checked with other techniques. Velocity nomograms were obtained for a number of mudslide cross-sections and the long-profile. The technique permits the accurate location of this boundary, since it is not necessary to extrapolate the results between measured points. 'Shape' electrical diaphragm pressure transducers were successfully used to monitor pore water pressures (section 5.4.2). These increased the accuracy of results because the effects of fissuring around and water percolation down the exterior of standpipes was eradicated (section 5.6.4). It was also possible to measure artesian pore water pressures during particularly wet conditions.

Field monitoring has been carried out on a time base not achieved by related studies (section 5.5). The results highlight the advantages to be gained from conducting such detailed investigations. Records of movement, pore water pressure and all climatic variables were kept on a five minute time base. This provided over 700 000 individual observations. Previous studies have failed to reach this resolution. The results suggest that to study, in detail, the interaction of variables governing geomorphological processes, frequent sampling is necessary. While it is impossible to present all the results from data sets as large as those collected for this study, significant events can be easily identified for further consideration.

Brunsden (1985) notes that problems frequently arise from monitoring different parameters on varying time scales. This has been overcome here. Not only have high frequency temporal records been kept but also many of the important variables have been recorded at identical points in time, on the same time scale.

The work presented here has extended previous studies in the following respects:

Brunsden & Goudie (1981) note that: "Lulworth Cove and its neighbouring bays are probably the most frequently visited, poorly described and least understood of all the famous geological and geomorphological teaching sites on the British coastline". This thesis has conducted a thorough investigation into the coastal geomorphology, elucidating much new information and providing a better understanding of the region.

Recent work on the mechanisms of mudslides has been conducted by Craig (1979) and Tan (1983). The former in particular develops a deeper understanding of mudslide behaviour on a short time scale and identified a stick-slip movement pattern. Tan (1983) complemented Craig's (1979) work and also developed a new 'in-house' cheap environmental data logging system. The current investigations have extended the high frequency temporal sampling framework adopted for these previous studies. Consequently, it has been possible to expand upon these investigations, confirming findings, reaching similar conclusions for a different material type and identifying previously unrecorded movement patterns.

Hutchinson and Bhandari (1973) proposed the theory of undrained loading. Suggestions have been made here of further developments to their model. These include: the presence of thresholds which have to be exceeded by controlling variables, for undrained loading at the head of a slope to be effective; the occurrence of one part of a mudslide causing undrained loading on lower sections of the track and the development of positive feedback mechanisms within individual systems.

The perturbation analysis proposed by Iverson (1985) for earthflow-like landslides seems to be applicable to mudslides. While not directly extending Iverson's work, scope exists for the lateral development of his theories with reference to the forms of mass movement discussed in this thesis. It is possible that the detailed measurements presented here may be used to validate the Iverson model.

As stated in the introduction to this thesis (section 1.3) a number of complementary studies have examined coastal slope instability in different parts of the British Isles (Brunsden & Jones, 1976; Bhandari & Hutchinson, 1982; Chandler, 1984; Craig, 1979; Hutchinson, 1974; Hutchinson & Bhandari, 1971; Pitts, 1981; Prior, 1975, for example). This thesis can be regarded as an extension to these studies in as much as a previously unexamined stretch of coastline has been investigated.

Studies of the structure of the Isle of Purbeck by House (1961), Melville & Freshney (1981) and Phillips (1964) have been enhanced by examining the geotechnical characteristics of the rock materials. Previous research has centred on a discussion of the changes in rock materials per se. This study has provided an extension to these investigations, by examining changes in material properties relative to structural trends.

Ultrasonic pulse velocity testing apparatus has been used to investigate the suggestion, propounded by Cooks (1981) that Dynamic Young's Modulus is potentially more useful for examining and classifying rock materials than has previously been suggested. Although Cooks' (1983) study refers specifically to drainage basin analysis and the effects of material strength and elasticity upon the denudational process and associated morphometric properties, he concluded that reliable inferences and predictions could be made on landform development, from data on the strength and modulus of materials. Results presented here support this. Moduli are a clear measure of material strength and also have a close association with other material characteristics including porosity and density.

Selby (1980, 1982) developed a technique for classifying rock masses for geomorphological purposes. This considers a number of parameters important to cliff stability, including compressive strength, joint orientation and evidence of water seepage through fractures. Relevant morphometric indices, such as the angle of the free face and total cliff height are also included in the scheme. Selby further extended his work by developing a weighting rating for each variable, depending on their relative significance in controlling the stability of the complete rock mass. This work has been used as a basis for developing a classification for the Portland Limestone and Wealden Beds, presented here. No such classification has previously been proposed for soft argillaceous materials. Due to the diversity of parameters important to the different materials exained, separate classifications have been developed. The variables included in any classification system and their class divisions, depend upon the way in which the classification is going to be applied.

Although seismic techniques have been used in the past to conduct subsurface investigations of mass movements, both Cratchley (1976) and Hutchinson (1982) stress the importance of further testing these techniques. This has been successfully completed here, although further studies ought to be conducted on different mass movement types.

A number of additional points are important. Changes to the coastal cliffs, the detailed consideration of present processes and the prediction of likely future changes have been examined. This study has provided an example of how such temporally diverse investigations can be successfully married together.

Predictive computer-based slope stability techniques must clearly be used with care. In some cases absolute stability analysis results were not as expected from the examination of other data collected as part of this study. While useful, relative, between site comparisons can be made, the specific results must be considered with caution.

It is possible to make recommendations for further research.

(i) The development of ultrasonic pulse velocity testing equipment, such as Grindosonic, will be advantageous not only to geotechnical investigations but also to a wide variety of geomorphological studies. The main advantage of this technique lies in it being a non-destructive test and it has considerable potential for further application.

(ii) Further studies of mudslides will provide additional insights into the mechanisms of movement and the characteristics of variables governing failure. Although detailed results have been presented here, it has only been possible to make recordings at a limited number of points on the slope. Work is still required in other materials and at other locations to corroborate these results. Also, by conducting monitoring at a number of points on one mudslide, more detail will be obtained about important spatial variable interrelationships within an individual system. This will permit further examination of the Hutchinson and Bhandari (1971) principle of undrained loading.

(iii) The clay mineralogy of the Wealden in the Isle of Purbeck needs to be examined by X-Ray diffraction. The results from this may be important due to the proximity of the mass movements to the coast and the dominance of onshore winds and subsequent inputs of specific ions (mainly sodium) to the landslide systems.

(iv) This stretch of Dorset coastline provides an ideal opportunity to examine the development of natural arches. Changes in structure from east to west have lead to the evolution of a number of these features displaying various fracture patterns. Further field studies, in combination with new techniques such as Finite Element Analysis, would greatly enhance an understanding of these landforms.

(v) Further investigation is required of the effects of pore water suction during extended dry periods on mass movements.

This thesis forms an important component in a sequence of ongoing research. Its contribution can be assessed both in terms of the details presented for this specific study area and also the conclusions which have much wider general application. The results provide a clear guide to where additional work is required.

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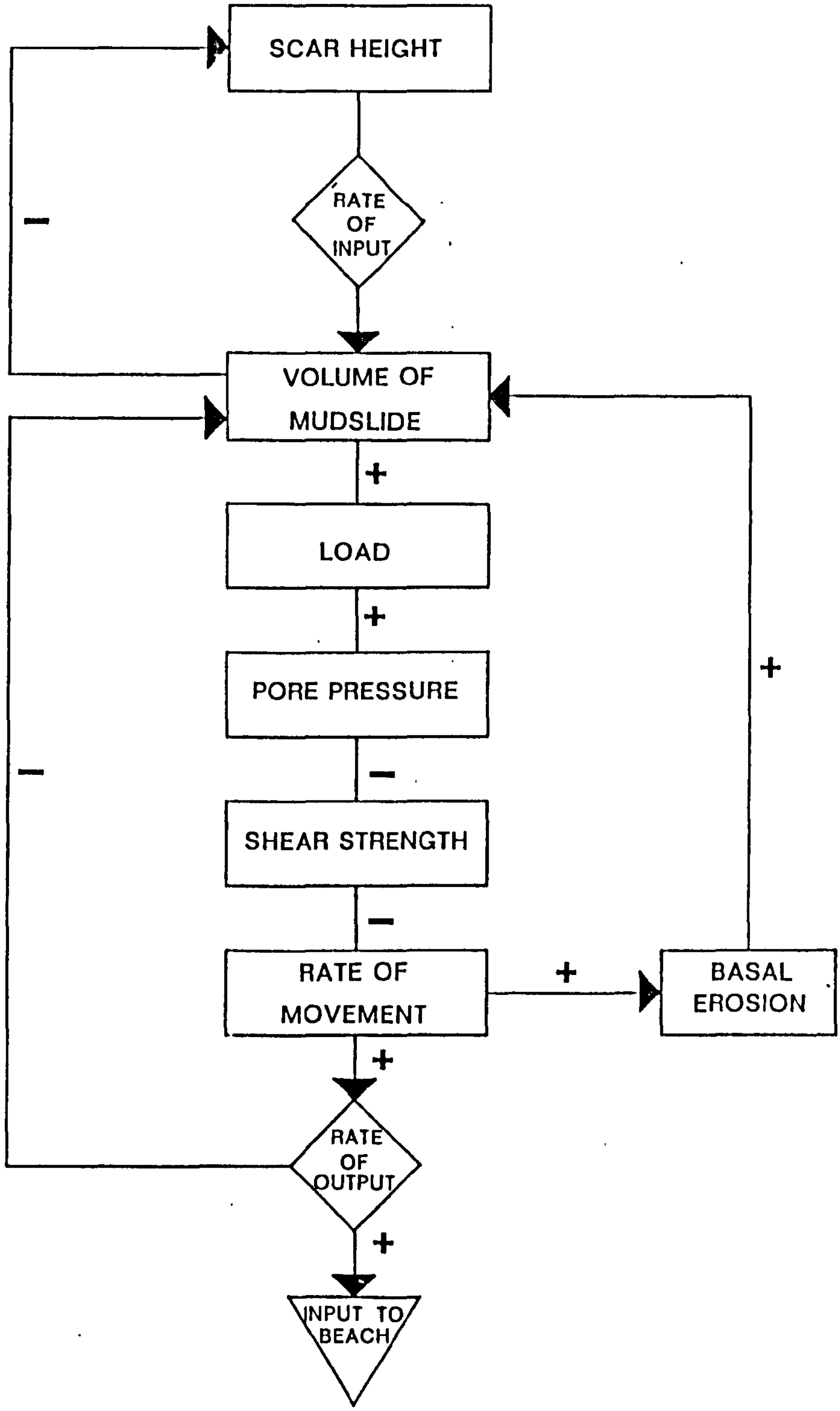


FIGURE 5.13

Summary of a mudslide as a process-response system

After Brunsden, 1973

Lulworth Camp
East Lulworth
West Lulworth
SCRATCHY BOTTOM
HAMBURY TOUT
BINDON HILL
Durdle Door
St.Oswald's Bay
DUNGY HEAD
Stair Hole
Lulworth Cove
FOSSIL FOREST
Bacon Hole
Mupe Rocks
Mupe Bay
Arish Mell
Worbarrow Bay
Tyneham
Tyneham Gwyle
WORBARROW TOUT
Pondfield
GAB CLIFF
Brandy Bay
Kimmeridge Bay
Kimmeridge Ledges
WHITEWAY HILL
PURBECK HILLS
Corfe Castle
Chapman's Pool
Hill Bottom Stream
EMMETT'S HILL
ST. ALBAN'S HEAD
Winspit
Seacombe
Dancing Ledge
ANVIL POINT
DURLSTON HEAD
Durlston Bay
PEVERIL POINT
SWANAGE
Swanage Bay
Ulwell
BALLARD DOWN
BALLARD POINT
Studland
Studland Bay
Old Harry Rocks
N
0
3km
Areas of cliff instability


804
805
813
805
822
805
802
801
818
802
825
802
826
799
813
798
833
798
817
795
827
795


FIGURE 3.19

The geomorphological map of the study area


SY 8480
SY 8580
SY 8680
SY 8479
SY 8679
SY 8779
Mudslide
Rotational Block With Back Tilt
Shallow Translational Slide
Translating Block
Shallow Surface Movement
Soil Creep
Rock Cliff
Rock Fall
Surface Wash
Cave
Natural Arch
Spring
Stream
Protected Stream Bank
Marsh
Surface Disrupted By Man
0
100m

FIGURE 2.2

Topography of the Isle of Purbeck

FIGURE 3.2

Changes in coastal cliffs identified from topographic maps

NOTE:

(i) Base for assessing cliff retreat is 1888 County series

(ii) Changes between 1888 and 1902 are shown in red

(iii) Changes between 1902 and 1955 are shown in black

(iv) Changes between 1955 and 1960 are shown in hachures

FIGURE 3.19

The geomorphological map of the study area